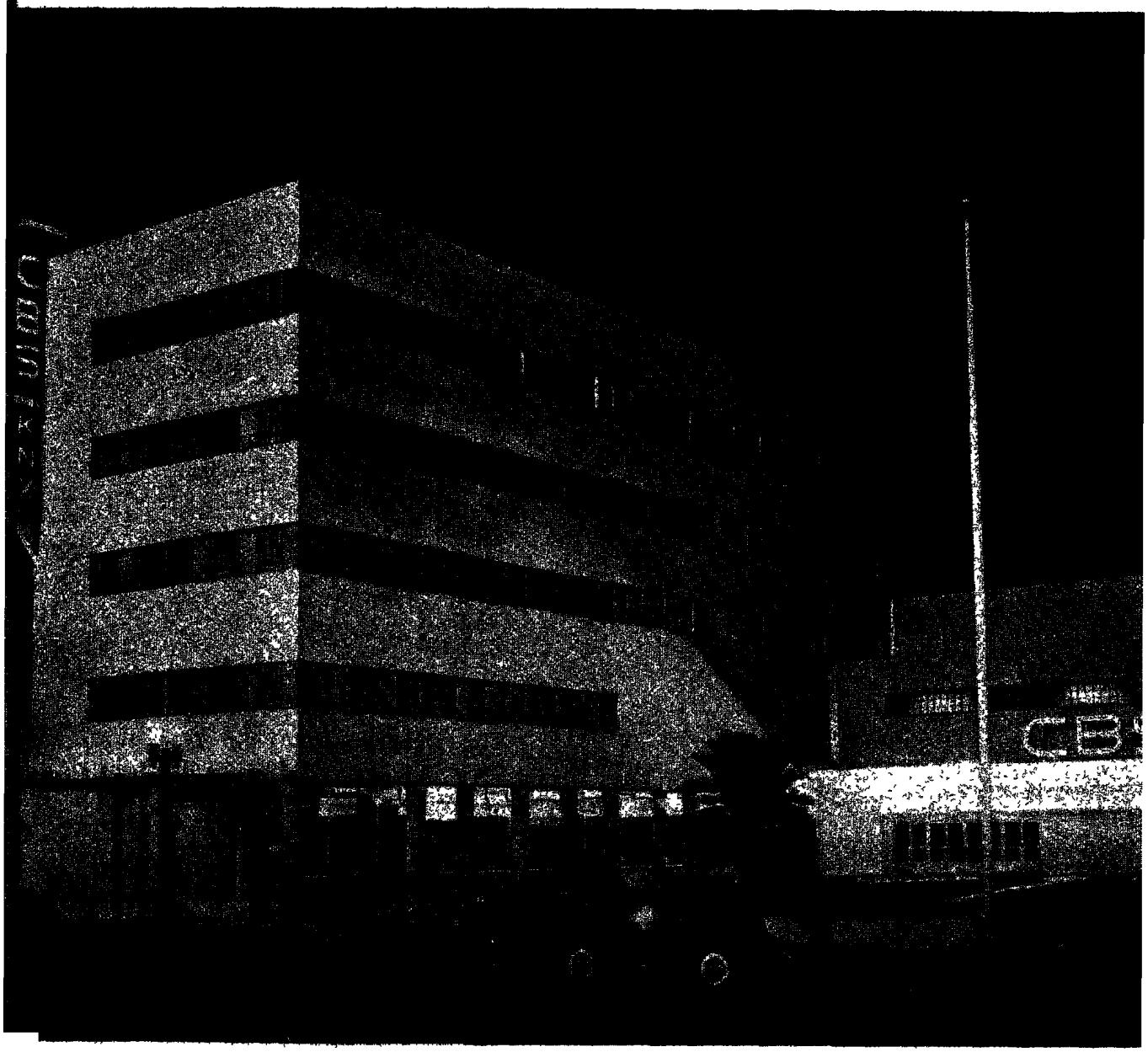
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The INDIAN CONCRETE JOURNAL

VOLUME XX SEPTEMBER 1946 NUMBER 9



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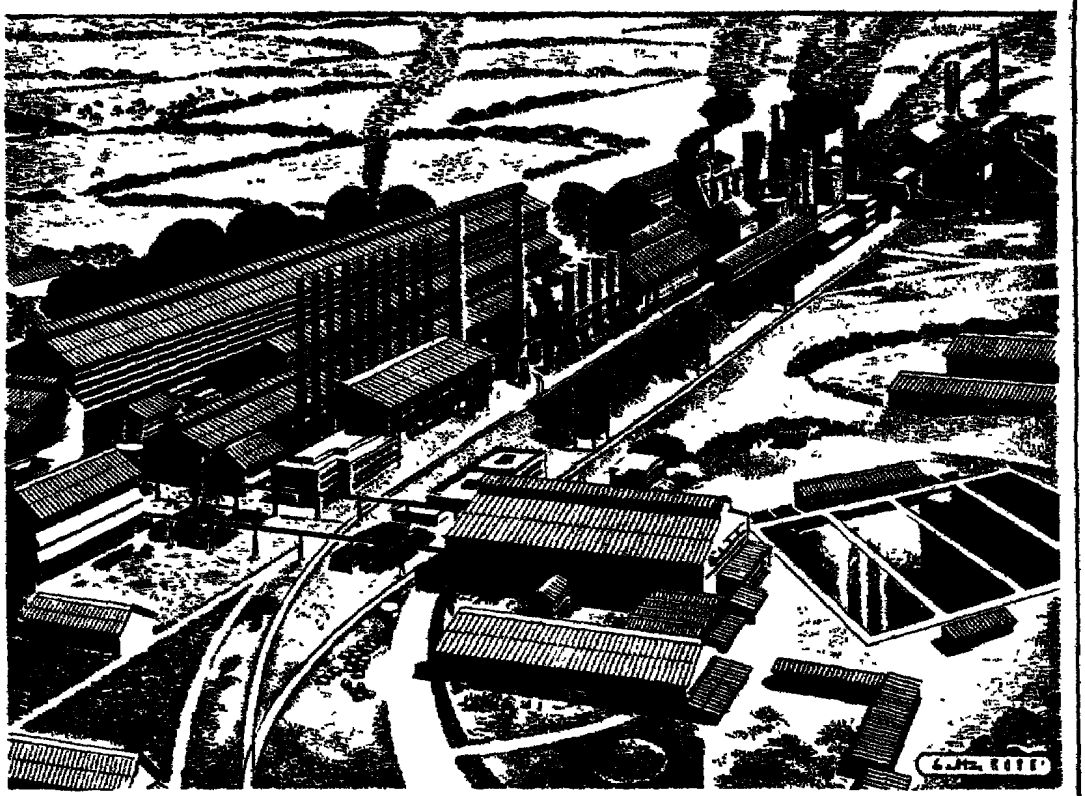
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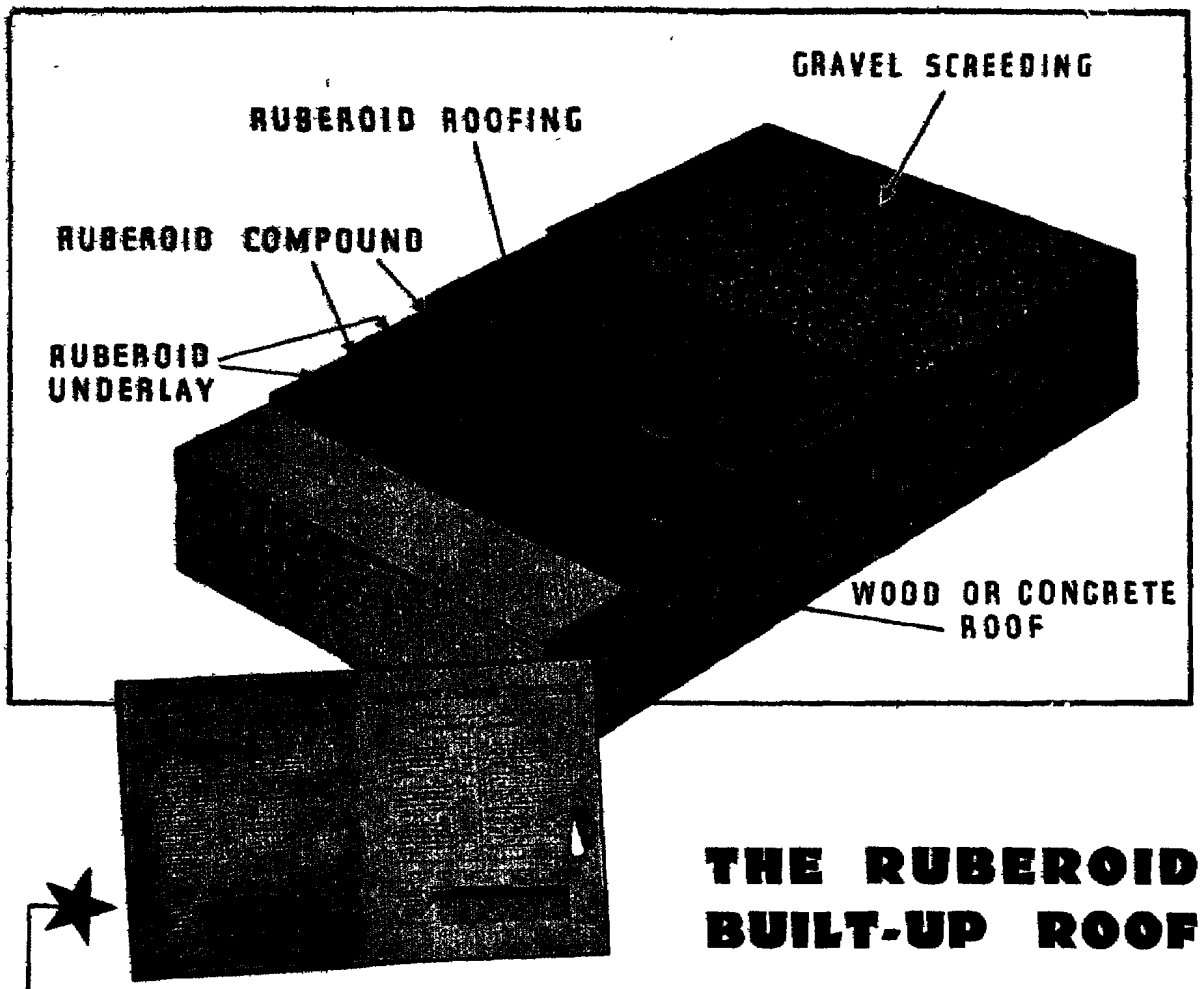
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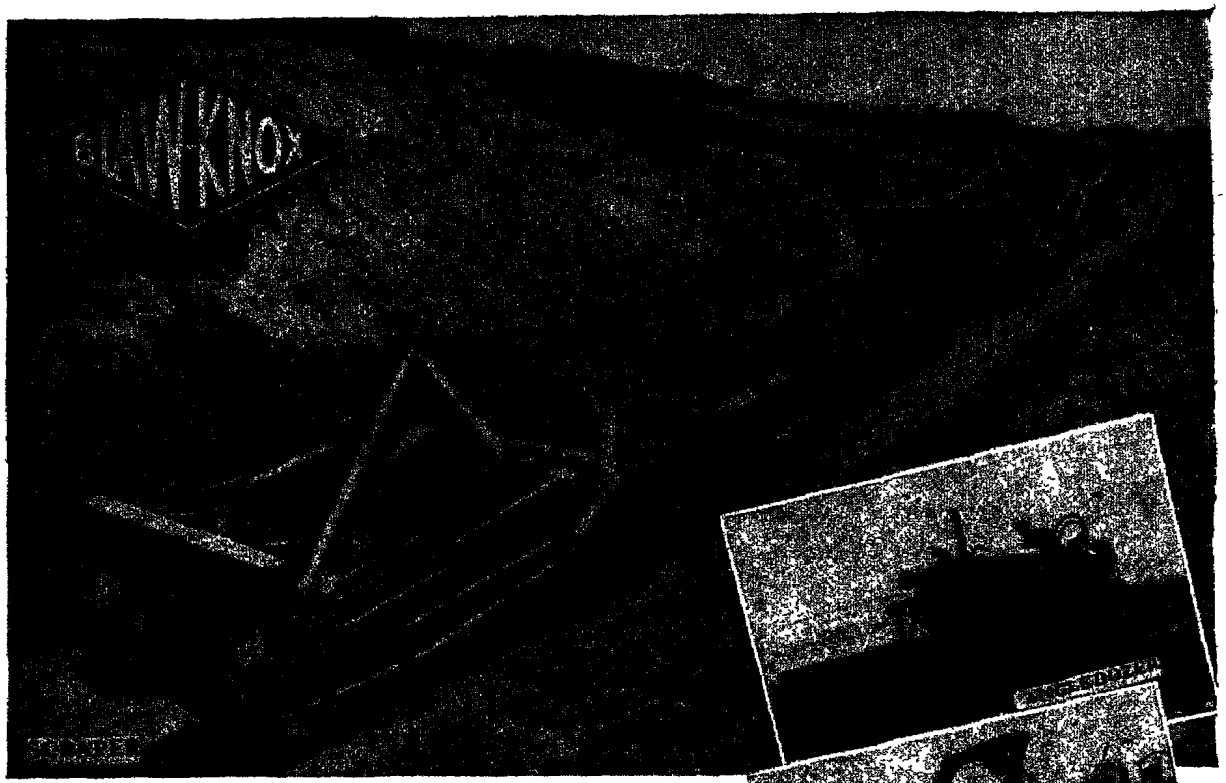
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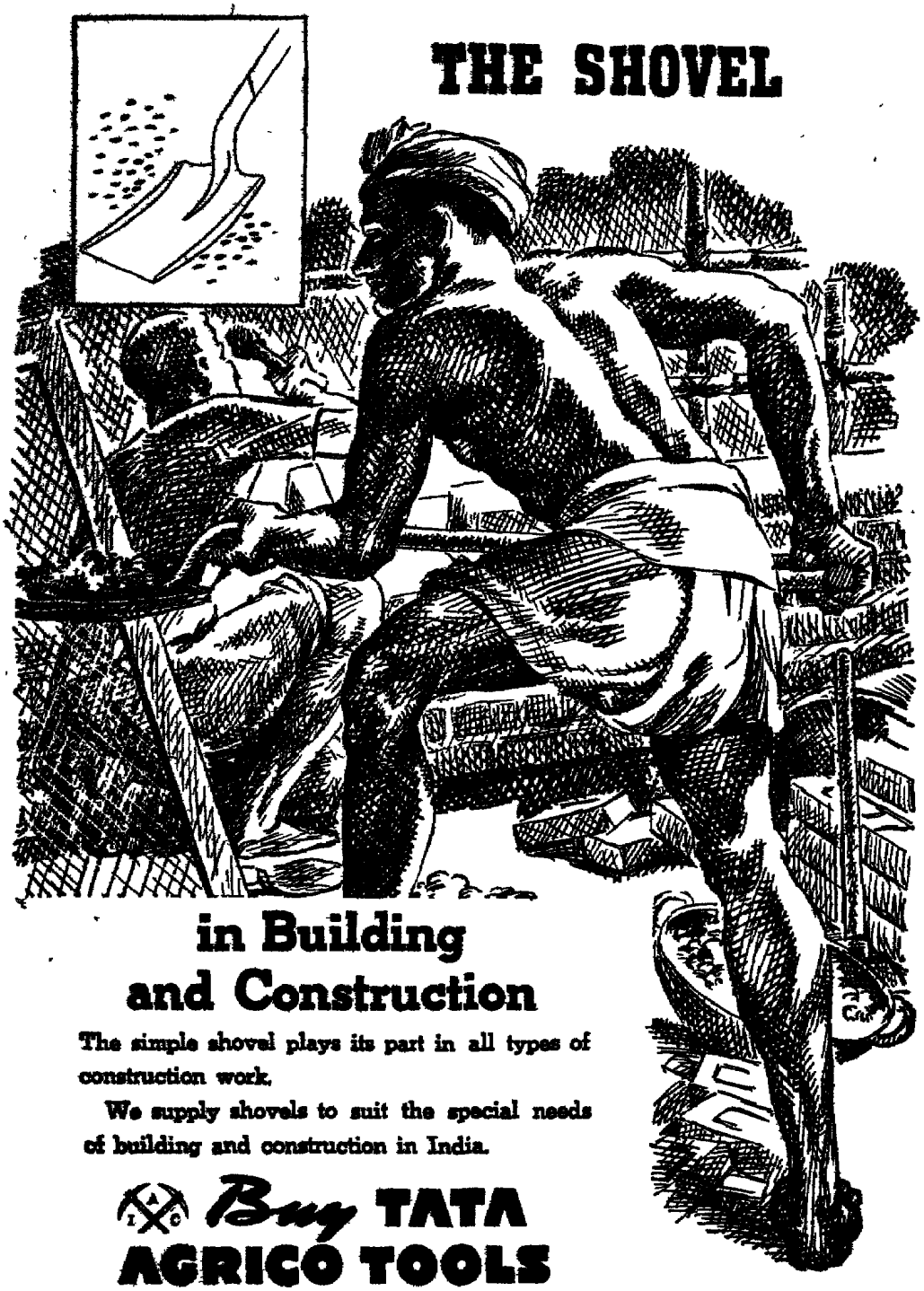


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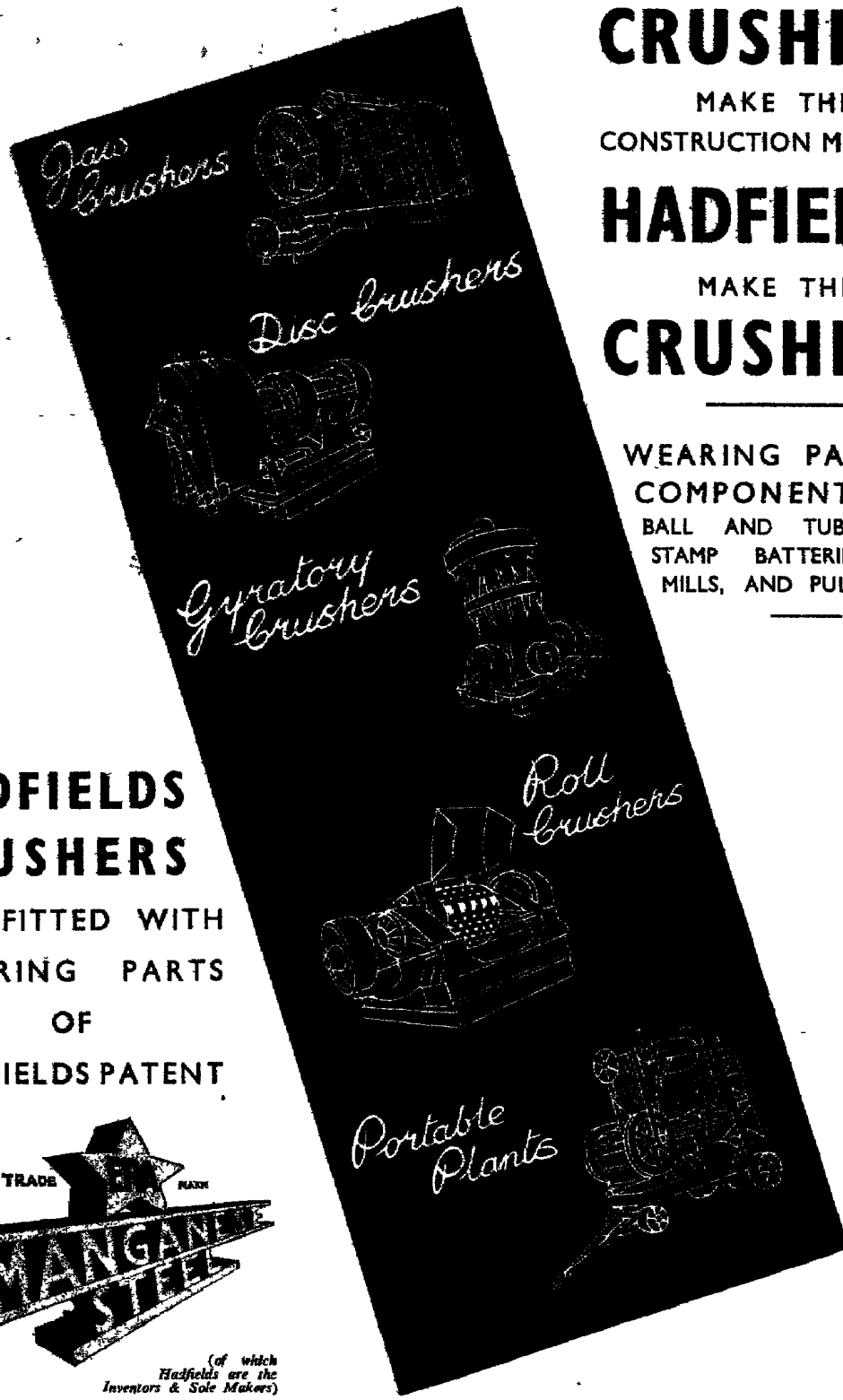
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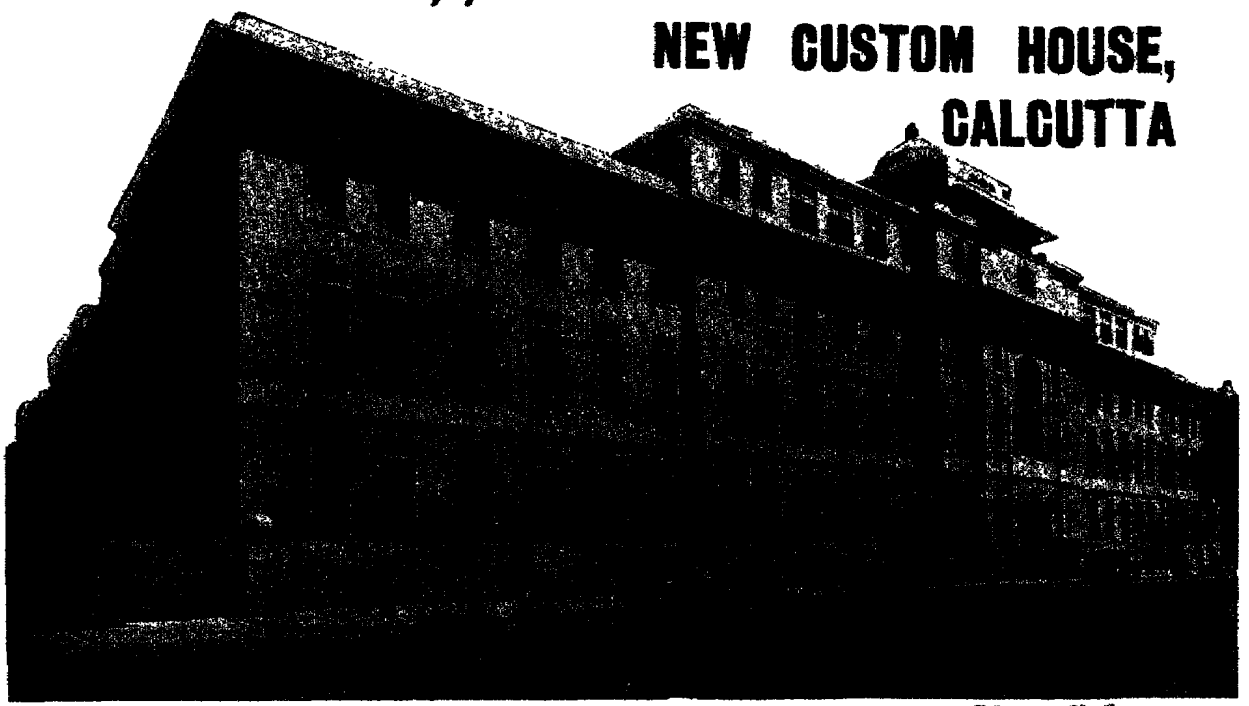
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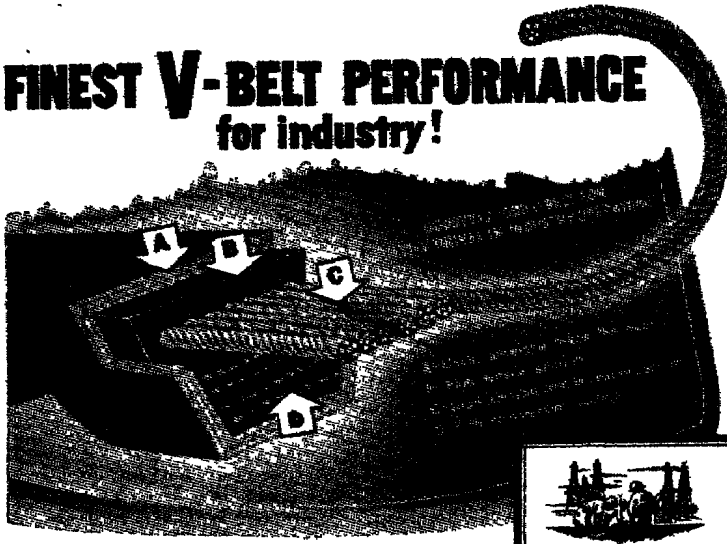
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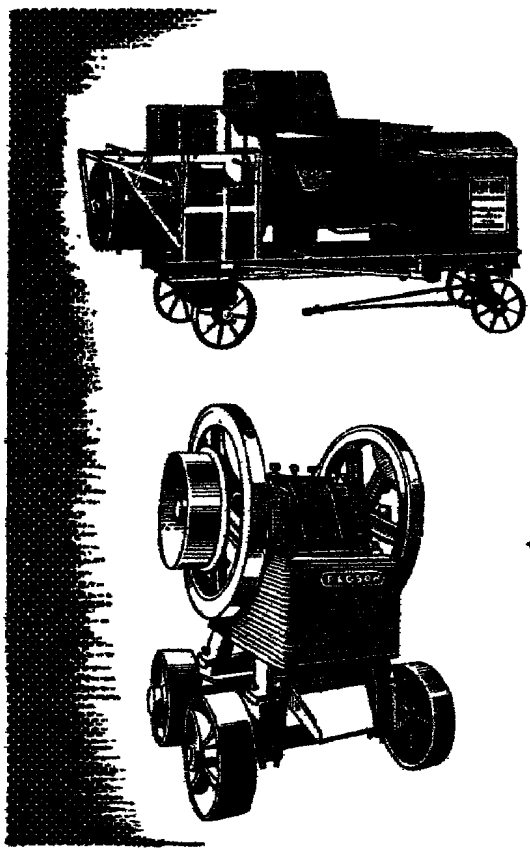
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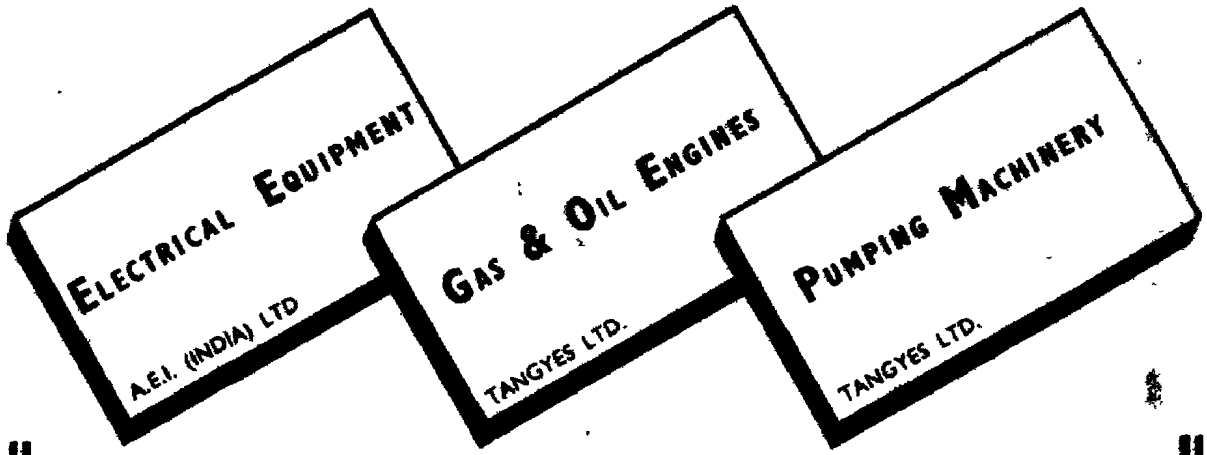
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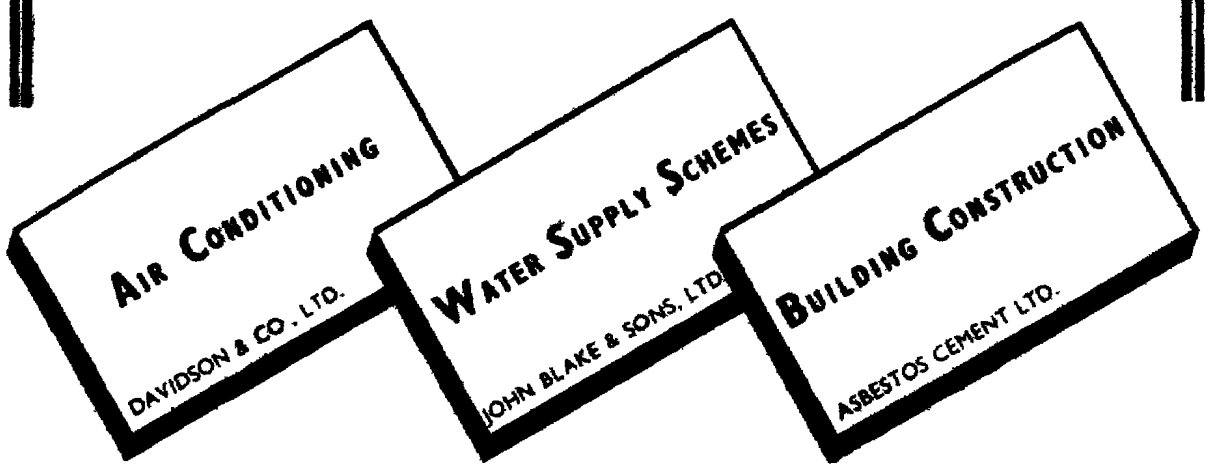
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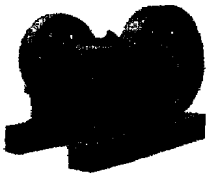
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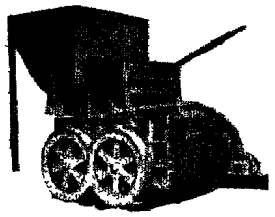
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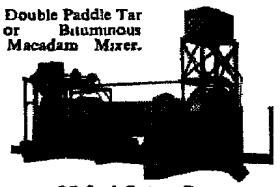
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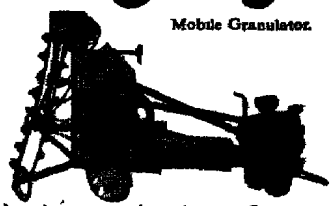
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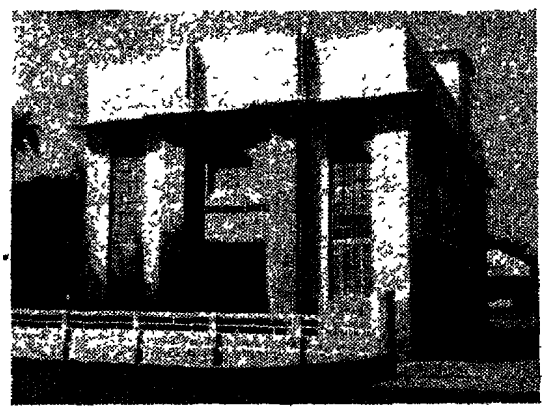
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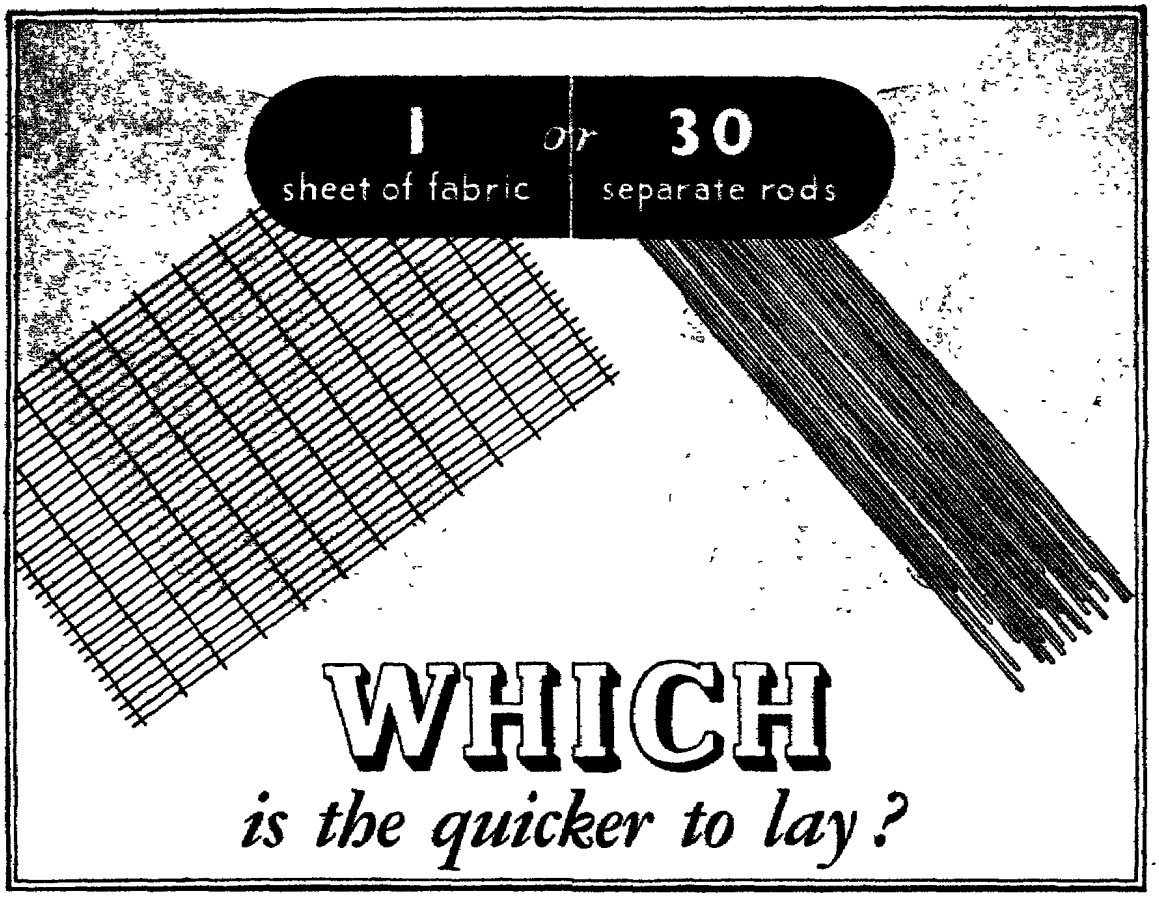
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REINFORCEMENT for CONCRETE



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is the quicker to lay?

The yield point of drawn wire from which B R C Fabric is constructed is more than twice that of rolled steel rods.

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EDITORIAL NEWS & NOTES



IRRIGATION AND HYDRO-ELECTRIC SCHEMES

ONE can hardly pick up a paper these days without seeing large type headlines describing some vast Engineering Project to control water or use it for the generation of power.

The more schemes that are thought out and brought to fruition the better, as there is a crying need throughout India for the surplus waters of the Monsoons to be harnessed for general use.

To bring a scheme to a successful finish it is necessary that every part fits for just as a clock needs all its parts working together in harmony before it will tell the time, so does an engineering work however large or however small require all the component parts to fit and work together to make a successful whole.

We read of the Rs. 55 crore Damodar Valley project to irrigate 760,000 acres in Bengal, the 17 crore Hydro-electric project in the Punjab known as the Nangal Hydro-electric and Irrigation Project. There is a 60 crore scheme for carrying out the Mahanadi Valley Multi purpose project in Orissa. Then there is the Marora Dam Scheme on the Nayar river in the United Provinces. There is also the 50 crore Kosi river dam project to control this restless river and provide irrigation and in addition there are all the National Highways to be constructed.

Each Province has its pet schemes from bunding to water supply from tube wells to aerodromes until the average reader must be completely bewildered with the vastness of it all and one's natural reaction is to say what on earth have our Engineers and Scientists been doing in the past or they may even ask where is all the money and talent to come from to make all this possible.

We are naturally amongst the last people to throw cold water on this vast engineering development work and we should welcome it all whole-heartedly but for an uneasy feeling that it is almost too good to be true.

We once heard an overworked District Engineer say that the man who made even so humble a thing as a well did more good to India than the average politician. Be that as it may, we do feel the time has come for getting down to brass tacks and, while hoping for great things, shall not be happy until we see even the smaller engineering projects at least in process of construction and fulfilment.

Water is obviously one of the main means of development of the country.

Billions of tons fall on to the land every year and it is nearly all wasted, for in a very short time the sun looks down on a barren and parched countryside longing for the water that has run to waste.

How can this waste be stopped. Cannot the Provincial Governments concentrate more on getting the villager to help himself. Generally speaking the agriculturist is no fool and knows what should be done but he is so poor and usually so undernourished that he has not the means or energy to get going.

Practical help in the shape of loans to villagers to work out their own salvation coupled with an instruction drive by means of lorries equipped with the necessary practical advisors on bunding, damming, well-driving, pumps, windmills and all possible devices for the conservation of water.

Let the Forestry experts explain the danger of soil errosion, the value of planting trees and the necessity for protection of young trees from goats and cattle.

Thus in addition and complementary to the great Engineering Works, let the masses of people who live in the 700,000 villages of India turn to and do their bit in and around their homes.

If these simple-minded people could be educated to help themselves by visual demonstration in practical methods of soil and water conservation, we should hear no more of famines and poverty as the soil of India with the help of a fraction of the water which now runs to waste could grow all the food and provide all the nutriment required for the millions who at present have to fight for a bare existence on a parched and thirsty land.

* * *

Book Review

"ROAD BUILDING AND EARTH MOVING MACHINERY NEWS"

We have just received the first number of "Road Building and Earth Moving Machinery News" published by Volkart Brothers who are importers of International Tractors, Bucyrus-Erie, Adams and other American Road Building and Earth Moving Equipment. Attractively produced and fully illustrated, this news sheet describes the full range of machinery that will be available from these manufacturers. Special articles deal with equipment for moving earth over short, middle and long distances, with the difference between Bulldozers and Bullgraders and with the new Buffalo Springfield 3-axle Tandem Roller. Road Engineers and Contractors will find this publication full of interesting technical information. A limited number of copies will be available to interested parties who should write direct to Volkart Brothers, Dept. 15, Ballard Estate, Bombay.

THE EFFECTS OF REVIBRATION ON THE STRENGTH OF CONCRETE

(The original work was submitted as part of the Thesis for M. Sc. (Eng.) of the University of London in 1939.)

By **N. N. PURANDARE, B.E. (Civil) (Bom.), M.Sc. (Eng.) (Lon.), D.I.C., A.M.I.E. (Ind.)**

(SYNOPSIS *As the use of Vibrator for compacting concrete is becoming a general practice, revibration cannot be avoided When the vibrator is used to compact a new batch of concrete by the side or on the top of a batch that has been placed some hours before, the setting concrete is disturbed. To study the effects, 6" cubes and beams were cast and were revibrated after intervals of 1 to 5 hours, and the failing loads were compared with the loads taken by not-revibrated cubes and beams The results indicate that far from any harm being done by revibration, there is a definite increase in its strength provided the concrete is revibrated during the period of initial setting but before any permanent set has started. Advantage can however be taken of the properties of revibration to get a more homogeneous mass of concrete and also for the manufacture of precast products as reconsolidation on revibration can be well controlled.*)

THE use of Vibration for compacting concrete is becoming general practice. The advantages of vibrating concrete need hardly be stressed here The report of the Committee appointed by the Institution of Civil Engineers to investigate into the effects of vibration, deals with them in detail, but there was hardly any work done to find out the effects of revibration, when these investigations were taken in hand The problem of revibration arises when a vibrator is used to vibrate a new batch of concrete by the side or on the top of a batch that has been placed some hours before and is undergoing the Set Now the question arises as to whether there is any loss of strength due to disturbing the concrete that is setting or has taken primary set If there is, then the use of vibrators in practice is harmful to the safety of the structures and should not therefore be recommended The recommendations that the concrete should be in position before its initial set and should not be disturbed afterwards, were drawn up before the mechanical methods of compacting concrete came into practice This problem arose when the author was carrying on the experiments to find out "the effect on the strength of Tee-beams of casting the rib and the flange separately" As the Tee-beams were cast in parts, the concrete in the rib being revibrated when the flanges were cast Research to investigate the effects of revibrations was carried out by the author in the Civil Engineering Laboratories of the City and Guilds College, London, under the supervision of Professor A J. Sutton Pippard and Dr. W T Marshall

Casting of the specimen and the observed effects of Revibration

The concrete was 1 2 : 4 mix by weight with water cement ratio of 0·55. The coarse aggregate was ¾ in Thames ballast, the sand washed river sand and the cement rapid hardening Portland It was mixed in a concrete mixer for 3 minutes The temperature of the room was about 65°F during the entire tests

The compression cubes were cast in 6" iron moulds on the vibrating table The beams 5½" × 3" by 3'-6" were cast in wooden mould The concrete in all these specimens was vibrated on the table for two minutes at the time of casting. The concrete was revibrated after some definite interval of time ranging from one to five hours To compare the results some specimens were also made of similar concrete but they were not revibrated. The specimens were taken out of the moulds on the next day, cured in water and tested after 7 days.

When concrete is of plastic or dry mix such as 55% or lower water-cement ratios in 1 2 4 mix, it looses its shining appearance due to the water on the top and looks dull and dark when left aside for about 10 minutes after mixing This concrete when put in moulds and vibrated soon gets its original appearance back The concrete in the moulds settles down, air is driven out, and when vibrated for two minutes, gives a mass of uniform texture with a surface easy for trowelling and finishing This concrete again takes its dull appearance after a short time

When the concrete is revibrated after one hour, the whole mass is 'fluidified', becomes plastic, there is further settlement but hardly any more air bubbles are noticed Excess of water appears on the top

When the concrete was revibrated after an interval of two hours, it took some time before it was fluidified Just after starting the vibrator, the dull thin crust on the top broke and after this the whole mass fluidified If the vibrator had been stopped before this had happened, hair-line cracks would have been left on the surface In this case also there was further settlement of the mass when fully revibrated The same effects were noticed when concrete was revibrated after 3 hours.

When the concrete was revibrated after four and five hours, the phenomenon of fluidification was not noticed. There were no apparent surface cracks, but there was a fine film of water on the top There was no further settlement

In every case the concrete was revibrated for two minutes It was finished a shade higher when it was cast so that after further settlement, the section remained the same as the non-revibrated concrete specimens

Method of testing and types of failure

The cubes as well as beams were cured in water and tested after 7 days. The cubes were tested in compression in the Riehle machine With every batch of revibrated cubes tested, cubes non-revibrated were simultaneously tested, so that their respective strengths be compared and percentage increase in strength of the former over the latter be calculated

The beams 5½" by 3" by 3'-6" were tested on a span of 3 feet and loaded at two points in the middle 8" apart giving a lever arm of 14" for Bending moment In the constant Bending moment zone an extensometer was fitted to measure the strain in the extreme compression fibre

In the case of cubes it was noticed that the ultimate crushing load taken by each of the revibrated cubes did not differ so much as the non-revibrated cubes The failures were similar and no segregation of heavier aggregate could be noticed in revibrated cubes

As the beams were reinforced 2 Nos ½" dia bars on the tension side only, all the beams failed in shear The loads at the first appearance of cracks and at failures for revibrated beams were greater than those for non-revibrated beams The failures were similar in all cases

Similar investigations

There was no work published on the subject of revibration of concrete when the experiments were started in March 1938, but shortly after, a paper appeared in the Journal of the American Concrete Institute on "Overvibration and Revibration of Concrete", by Messrs. Tuthill & Davis The tests were carried out on concrete cylinders in compression and for bond stress, and the results were a definite gain in strength of 25%, when revibrated after an interval of from two to five hours. It was suggested that it may be due to the reduction of voids as the mass settles on revibration

Probst writes, "Transport over long distance may lead to an increase in strength if care is taken to avoid segregation by some system of stirring". He wrote this before vibrator was known. It shows that he had some ideas as to the loss of strength due to overvibration, and the beneficial effects if concrete is vibrated while it is setting.

Mr. Johnson of Penang read a paper on "Delayed placing of concrete", and

justifies the increase in strength by 25% to the lowering of the water-cement ratio due to the heat set up by the hydration of cement and increasing the evaporation of water in concrete. The abstract of the paper was published in "Concrete & Constructional Engineering Journal" in September 1939.

Theories relating to Setting and Hardening of Cement

The setting and hardening of cement is very complex It arises not only from the presence of several different compounds, all of which undergo hydration, but also from the physical nature of their hydration products.

Cement is employed in the form of powder, and when mixed with appropriate quantity of water and some suitable aggregate, yields at first a plastic mass, capable of filling and taking the shape of some confined space in concrete work At first the mass looses its plasticity, and becomes more or less friable, so that when it is revibrated or remoulded, the plasticity is only partially restored ; after a certain time, consolidation takes place, the mass increasing in hardness, a stony texture is finally obtained. These two stages are distinguished as the processes of Setting and Hardening

The mechanism of these changes cannot be definitely stated. The production of a coherent mass from powdered solid and water may take place in different ways

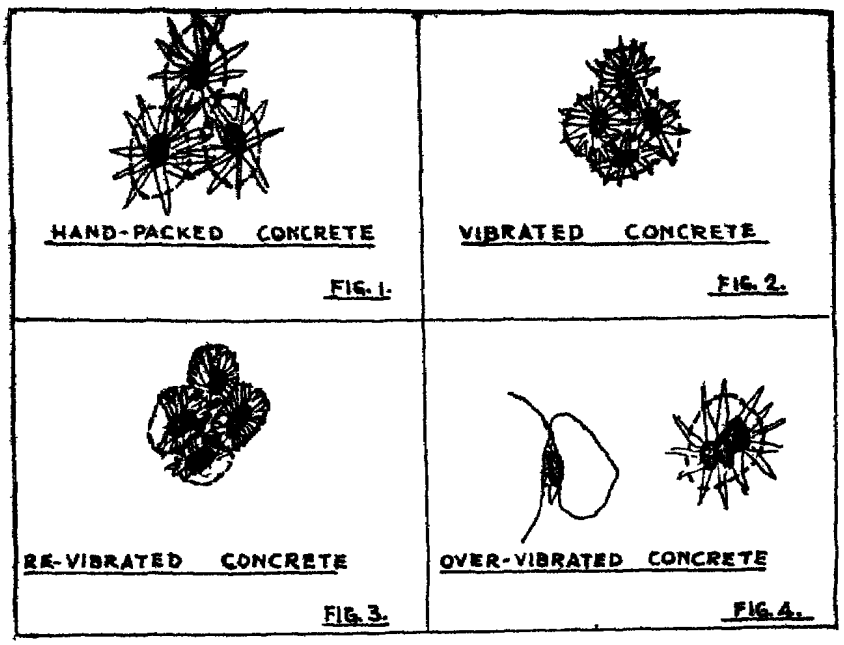
(*a*) The crystallisation of some of the constituents producing a mass of interlaced crystals

(*b*) The formation of colloidal mass or ' gel '

(*c*) The combination of *a* and *b* in presence of water.

Two main theories have been advanced to explain the hardening of cement The first and the oldest is the crystalline theory put forward by Le Chateher, which ascribes the development of cementing properties to the growth of interlocking crystals This theory explains well why excess of water reduces the strength of concrete With excess of water the crystals formed are long with less interlocking, and if it is said, that these crystals, which are very slender, transfer the stress, then as will be naturally expected, they cannot take as much load as the shorter crystals formed when less water is added

The second theory, which was first propounded by W. Michaelis, is the Colloidal theory According to this theory, a set cement is a colloidal gelatinous mass which hardens as it dries and shrinks The reaction of water with cement produces hydrated compounds in solution which, owing to their insoluble nature, become highly supersaturated. As Michaelis puts it, " After the decomposing and dissolving influence of water upon freshly gauged cement has lasted for some time, the supersaturated solution surrounding the cement grains coagulates This is the point of time when water disappears on the cement pats and when setting proper has taken place ". The gelatinous mass was supposed to consist mainly of a hydrated Calcium Silicate with some Aluminate, and to contain distributed in it the crystals of Calcium



Hydroxide and other Hydrated Aluminates The hardening was due to the gradual elimination of water from the gel , this process was considered to be assisted by the formation of the Hydroxide and the Aluminate compounds and by the withdrawal of water from the gel layer to hydrate the inner unhydrated core of the grain

The gel formed in the setting cement is due to hydration of Tricalcium Silicate When finely ground Tricalcium Silicate is mixed with water, hydration commences within few hours , crystals of Calcium Hydroxide soon appear while gelatinous hydrated Calcium Silicate is formed round the original grain. Complete hydration cannot be obtained unless the Tricalcium Silicate is very finely divided and the mix is reground at interval to expose fresh surfaces to the water If this is not done, a product is obtained showing unattacked cores of Tricalcium Silicate surrounded by a layer of gel which, being impervious to water, renders further attack slow It appears therefore that the setting process is a combination of crystallisation and coagulation This may be an explanation of the increase in strength due to revibration

The series of figures that are given here, are intended to give some idea as to what may be happening due to revibration and the increase or decrease of strength thereby

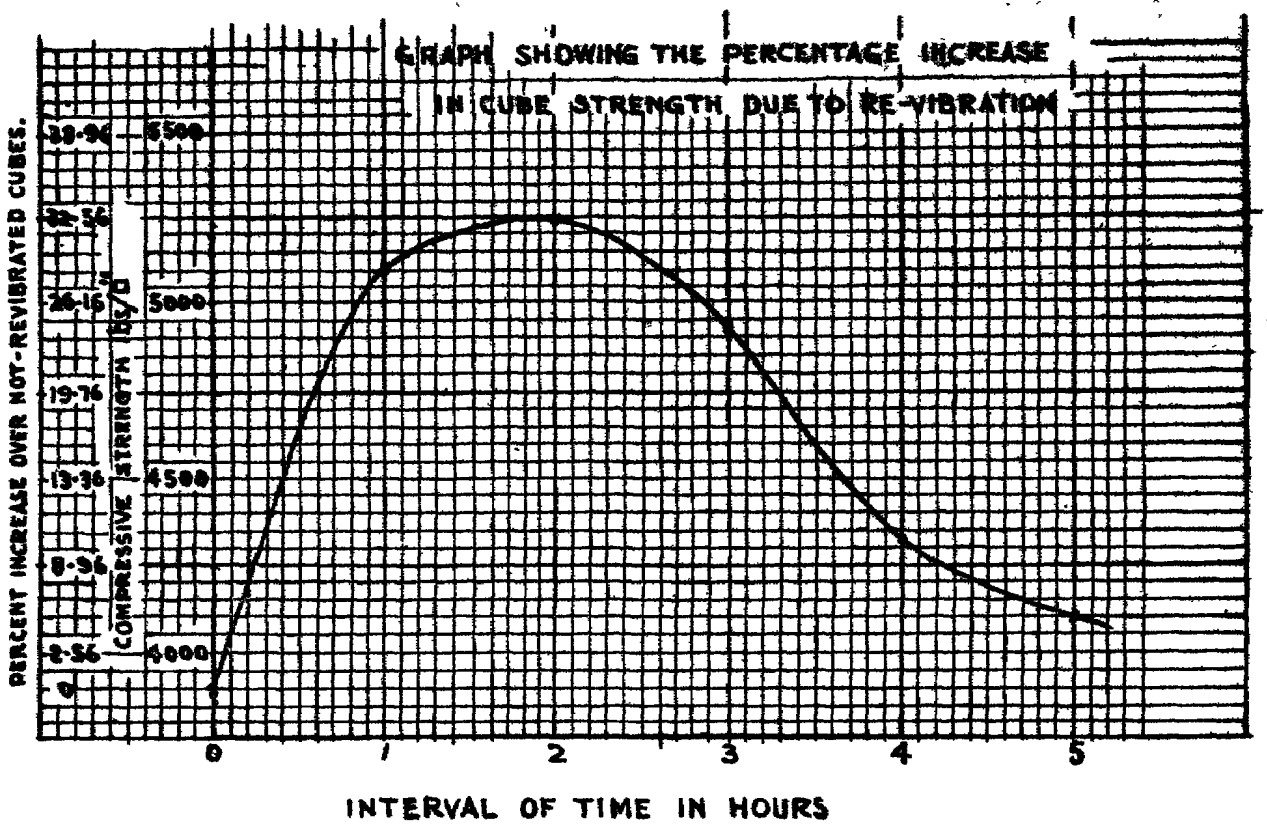
Figure 1 gives what may be the general formation of crystals and gel in concrete that is consolidated by hand The crystals formed are long as wet mix is required ; and the gel surrounding the ' core ' of unhydrated cement particles may be so far apart as to give space for air and excess of water. When the concrete is vibrated at the time of casting, there is a better consolidation, the cement particles are closer together, the crystals are short, and the gel fills up all the voids But there is still a large amount of unhydrated core of cement

When the concrete is revibrated before it has set, the crystals are broken and each cement particle has a toothed surface due to broken crystals These particles are better wedged due to these teeth The ' gel ' is also broken and water can reach the inner unhydrated cement core More tiny crystals are formed in the space left and the ' gel ' covers and fills up all the remaining voids. This gives a better consolidated concrete and most of the cementing property concrete is used as hardly any unhydrated particles are left

When the concrete is revibrated after it has taken the set, the crystals are broken up but as the concrete is not plastic enough, there cannot be any rearrangement of particles and therefore advantage cannot be taken of the toothed surface As the ' gel ' has grown in thickness, it cannot be broken up, though it is cracked, and water cannot reach the unhydrated core. No more crystal or gel is formed and so the concrete remains cracked inside, which results in loss of strength When concrete has taken a permanent set, and then it is vibrated, the gel has hardened enough to take the vibrations without cracking, and so there is no loss or gain in strength

When the concrete is overvibrated at the time of casting, the loss of strength may be due to

(1) the settling down of heavy aggregate, and silt in sand with the cement

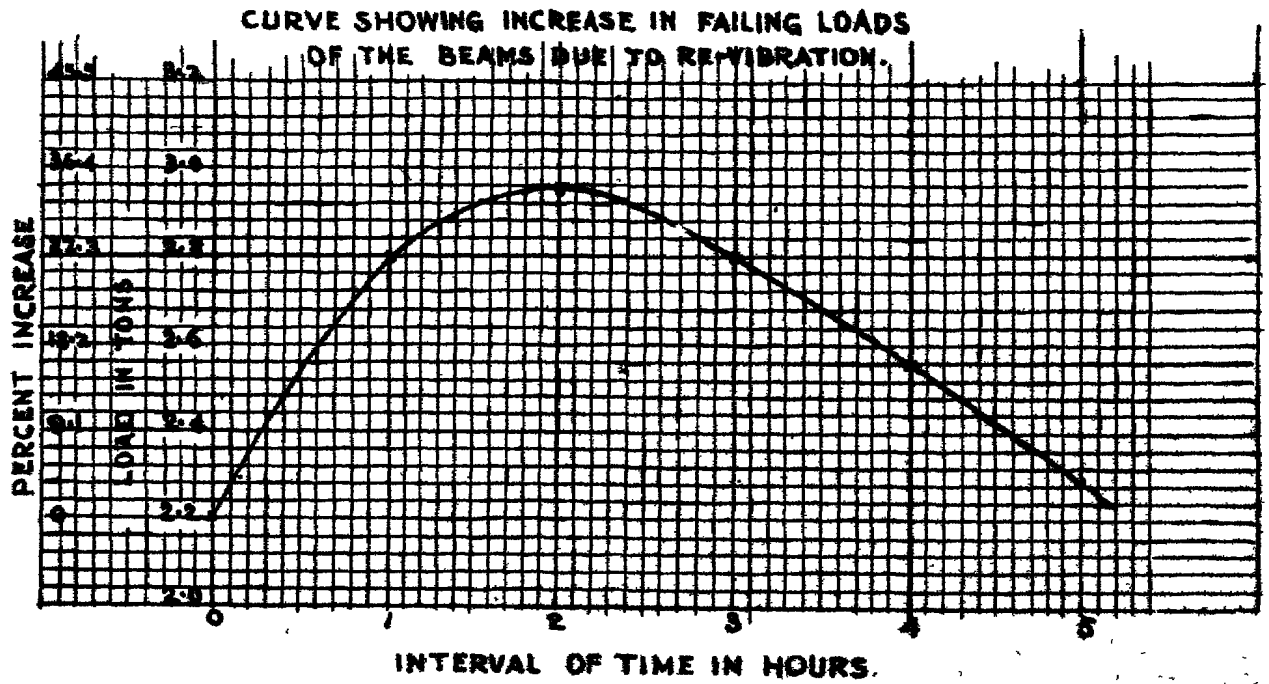


coming up to the top with water ; and

(2) trapping up of cement particles been heavy aggregates leaving no space for growth of crystals or gel It may be even two or more cement particles brought so close together that no space is left for hydration and the growth of crystals and gel, as the water is squeezed out

Conclusions and Recommendations

From the results of the experiments, it may be concluded that there is no harm done to the concrete if it is revibrated before it has set ; as a matter of fact it gains in strength, and where concrete is placed in layers, there is a more homogeneous and uniform concrete formed after this operation. Since the increase or decrease in strength depends upon the amount of set the concrete has taken



TABLES SHOWING THE EXPERIMENTAL RESULTS OBTAINED ON BEAMS, CUBES & TENSION SPECIMENS

Item	Compressive cube strength lb /in² (7 days)	Percentage increase over not-revibrated cubes
Not-revibrated cubes 6" × 6"	3900 0	
Revibrated after 1 hour	5119 0	31·3
" " 2 hours	5239 1	34 3
" " 3 hours	4933 3	26 5
" " 4 hours	4312·0	10 6
" " 5 hours	4100·0	5·12
" " 24 hours	3906·0	Nil

BEAM 5½" × 3" × 42" with 2, ⅜" Dia. bars.

Item	Ultimate Load (7 days) Tons	Percentage increase over not-revibrated beams.
Not-revibrated	2·2	
Revibrated after 1 hour	2·8	27 3
" " 2 hours	2·975	35 2
" " 3 hours	2·8	27·3
" " 4 hours	2·572	17 0
" " 5 hours	2·30	4·55

TENSION SPECIMEN, Area 2½" × 3" = 7·5 Sq. in

Item.	Ultimate tensile strength (7 days) lb /in.²	% increase over not-revibrated specimens
Not-revibrated	224·0	
Revibrated after 1 hour	284·1	27 1

before it is disturbed, and the mechanical possibility of putting it together, it must be noted that the concrete should not be allowed to be disturbed after a certain time This limiting time depends upon the type of cement, and its setting time, the water-cement ratio and the temperature of the air The other factor that plays an important part is the type of vibrator and its efficiency to fluidify and reconsolidate the concrete Though the question of amplitude of vibration was not touched due to practical difficulties, it was observed that the concrete could be very easily 'fluidified' when it was revibrated within two hours, while afterwards the mass became stiffer and took more time This may show that vibration may not always be sufficient to consolidate the concrete disturbed after two hours, since the amplitude of vibration would be much reduced due to the new position of the vibrator on the job

Revibration of concrete in a big structure to get a stronger concrete cannot be recommended as it would involve lot of time and labour and would thus be uneconomical Advantage can however be taken of the properties of revibration to get a more homogeneous mass of concrete by allowing the vibrator to penetrate to the lower layers This would be very useful in case of mass concrete dams as vibration will effect in a better bond and homogeneous concrete near the boundary of layers It can also be recommended for the use of precast products as reconsolidation on revibration can be well controlled.

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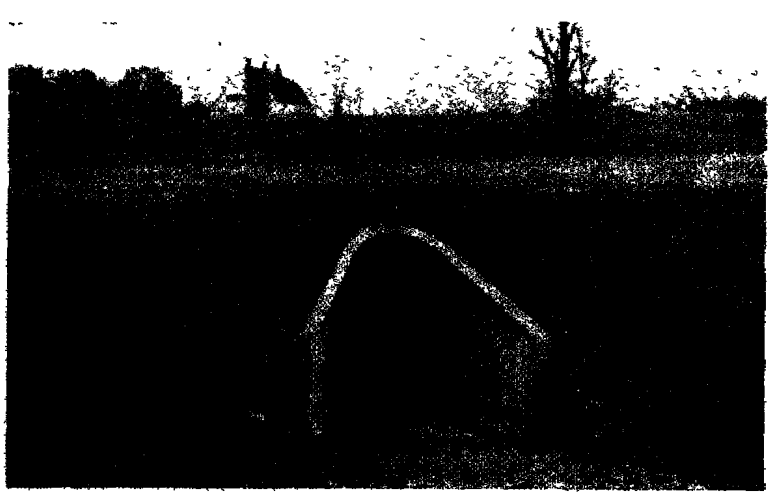
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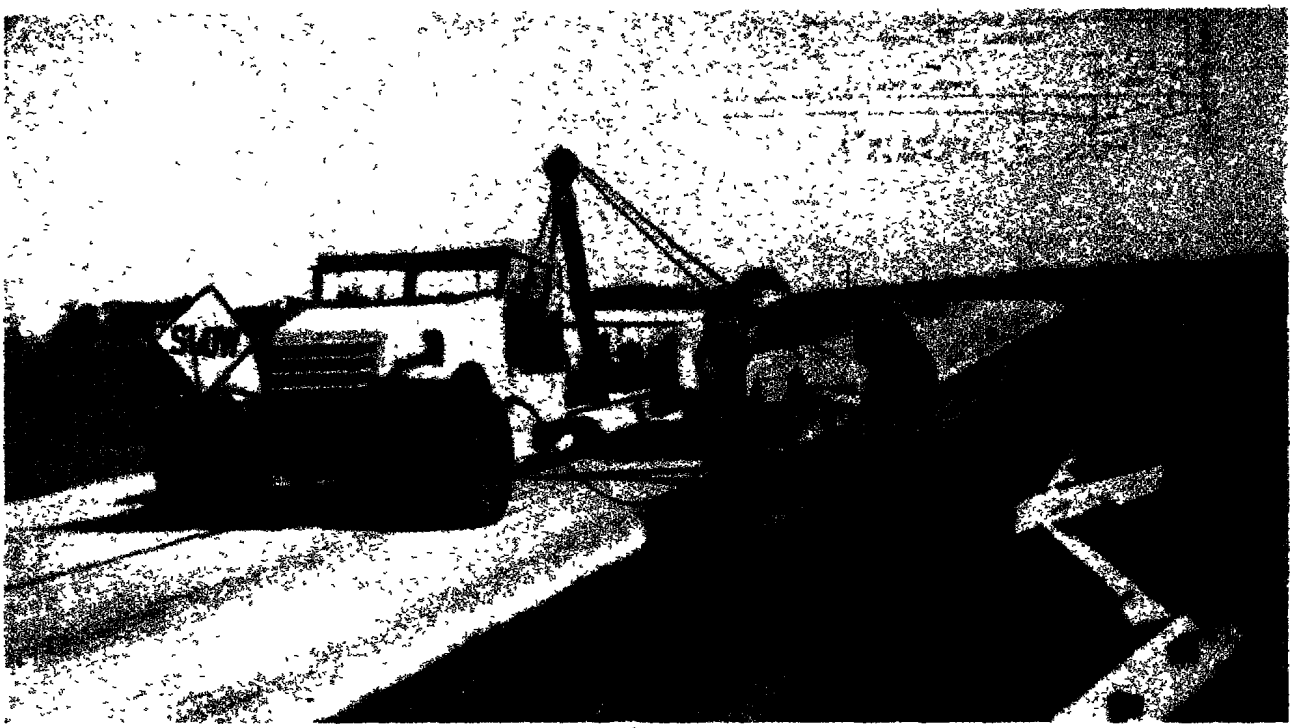
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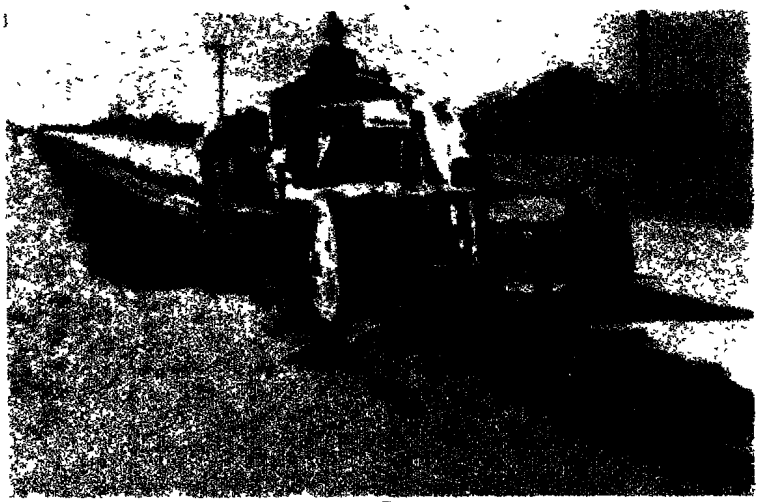
FIRST FLARED-END PRECAST CONCRETE CATTLE PASS goes into service on section of Minnesota Trunk Highway 100 constructed by A. Guthrie & Co., contractor, St Paul, just north of Twin Cities. Manufactured and installed by Elk River Concrete Products Co., Minneapolis, pass is 68 ft long and consists of eight 6-ft long sections and two 10-ft. long sloping ends Inside dimensions are 4 × 6 ft. If highway is later widened, tunnel can be extended in length without loss of materials.—("*Construction Methods*")



1

Special Equipmei

Strips Along Sides



2

1 FINE GRADING is done by cutting blade on form-setting machine mounted on side of truck. This rig moves forward as each form section is set and holds steel form at correct elevation and alignment while it is anchored in place with stakes (Sketch) WIDENING of existing 18-ft. pavement to 24 ft is done with 3-ft. concrete strips along each side. Old pavement is undercut and supported on shoulder of widening strip, which is 14 in thick at joint and 6 in. at outer end.

2 SPECIAL BLADE on motor patrol grader does rough excavation for widening strip and undercuts existing concrete slab.

3 SPECIAL MACHINE strikes off and finishes concrete Hopper receives ready-mixed concrete from trucks and screw conveyor carries concrete to subgrade. Finishing machine has screed and vibrator tamper.

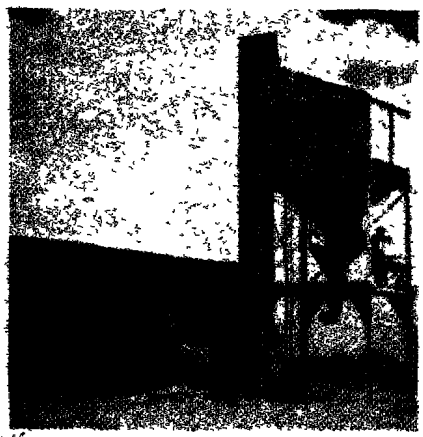
4 JOINT FILLER is 3-ft. long plank notched at both ends to permit passage of wheels of finishing machine Cutouts are replaced in plank after finishing machine has passed and dowel alignment is corrected before concrete is placed.



3



4



5

5 AT BATCHING PLANT railroad car is spotted alongside hopper into which bulk cement is deposited and raised by inclosed bucket elevator for discharge into bin.

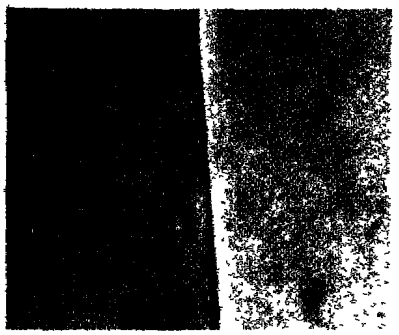
6 CAREFUL EDGING of new concrete at junction of old slab is essential to prevent overhang and subsequent ugly spalling.

7 LIFTING OF REINFORCING BARS to proper position is done by this device attached to front end of finishing machine

8 STEEL REINFORCEMENT, in form of longitudinal rods and transverse U-bars, is placed on subgrade ahead of concreting. Groove under existing slab is formed to receive concrete underpinning of strip

9 WIDENING STRIP IS HERE SHOWN completed along one side of existing concrete pavement.

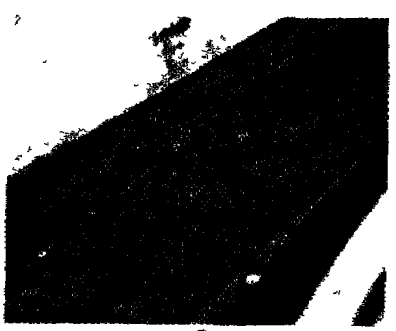
10 CURING OF CONCRETE is done by spraying on newly finished surface colourless waterproof membrane by Hunt process



6



7



8

Places 3-Ft. Widening Concrete Pavement

TEXAS IS WIDENING hundreds of miles of its old 18-ft. concrete pavements with 4 to 6 ft of new concrete slab With their customary ingeniousness, Texas contractors have developed special equipment for this work, which has materially reduced costs and, during the war years, made work possible in spite of labour shortages

Most of the widening has been a 3-ft wide strip on each side of the old pavement, but in some places all the widening is put on one side Widening all on one side is more economical from the standpoint of pavement construction, but often requires widening of structures and the use of newly placed subgrade, while the narrower widening placed on both sides, uses the old shoulder as subgrade and may not require widening of bridges and culverts.

A 25-mi. project from Edna, in Jackson County, to El Campo in Wharton County is typical. The existing 9-6-9-in. concrete pavement was 18 ft wide and a 3-ft. strip of concrete was placed on each side to make the new width 24 ft.

Thickness of the widening strip is the edge depth of the old slab plus 5 in , or 14 in , at the edge of the old slab, sloping to 6 in at the outer edge of the widening This 5-in depth is underpinning, extending 4 in back under the old slab Some widening was built without this underpinning, but was not satisfactory, particularly on highly expansive soils, as the widening was often lifted above the old slab, or separated from it, with volume changes in the underlying subgrade

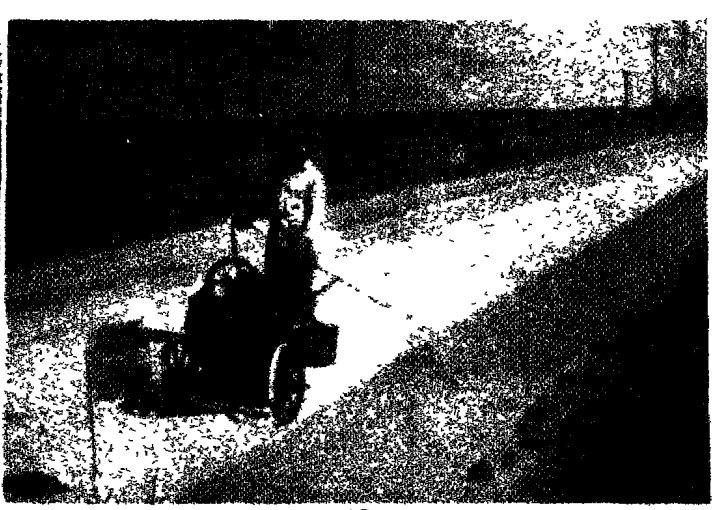
Each 3-ft widening strip is reinforced, as illustrated in the accompanying cross-section, with three longitudinal ½-in round deformed bars and one ½-in round transverse U-bar, 5½ ft long which extends under the edge of the old slab at intervals of 4 ft. Expansion joints are placed in line with the expansion joints in the old pavement and have three ¾-in round smooth dowels in each 3-ft strip Badly broken areas in the old pavement were replaced with new concrete ahead of the widening

The bid price, which includes concrete, steel, and the necessary excavation, was $2·85 per sq yd Construction was completed early in 1946

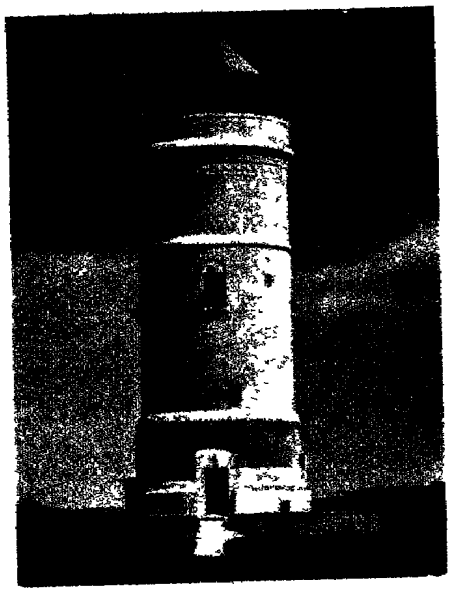
Cagle & Proctor, of Houston, were the contractors and work was under the direction of M. B. Hodges, district engineer, Texas State Highway Department — *(With acknowledgments to "Construction Methods")*



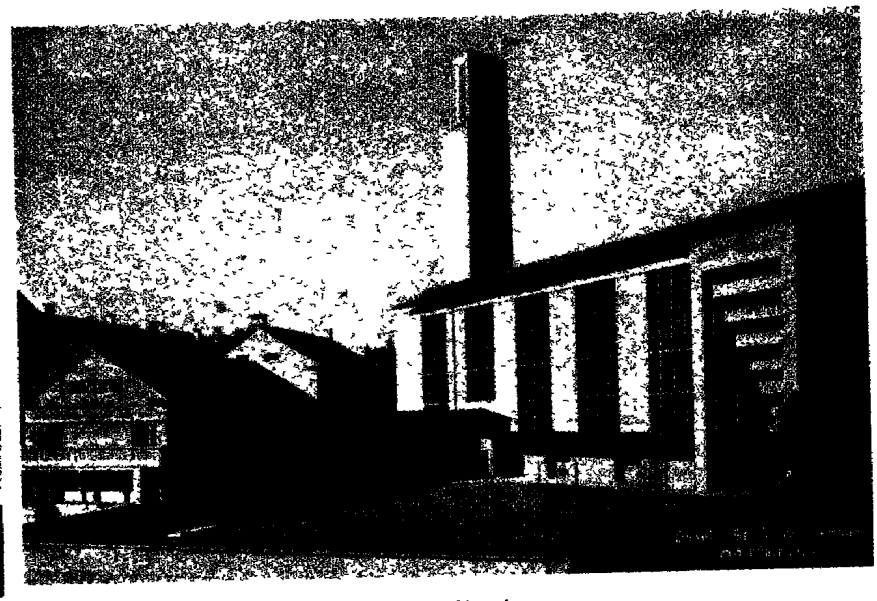
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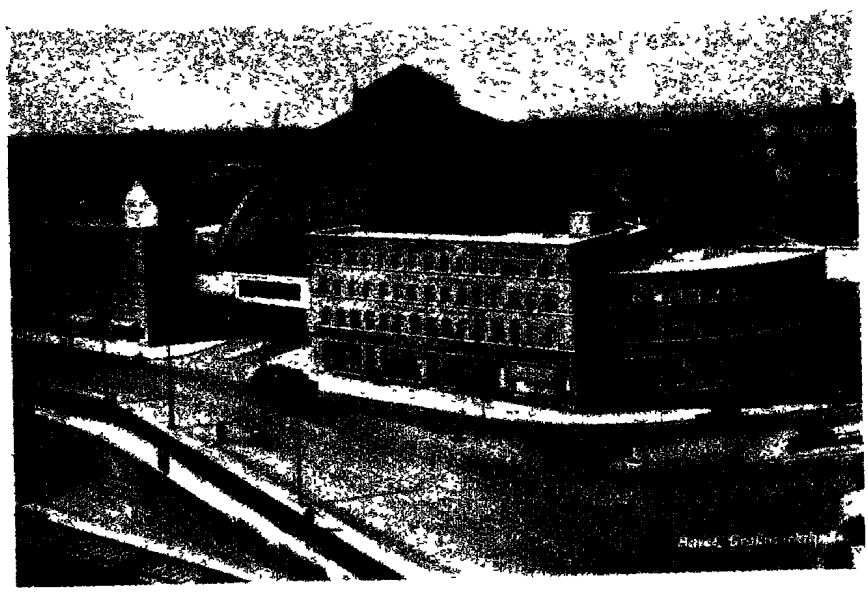


Water Tower.

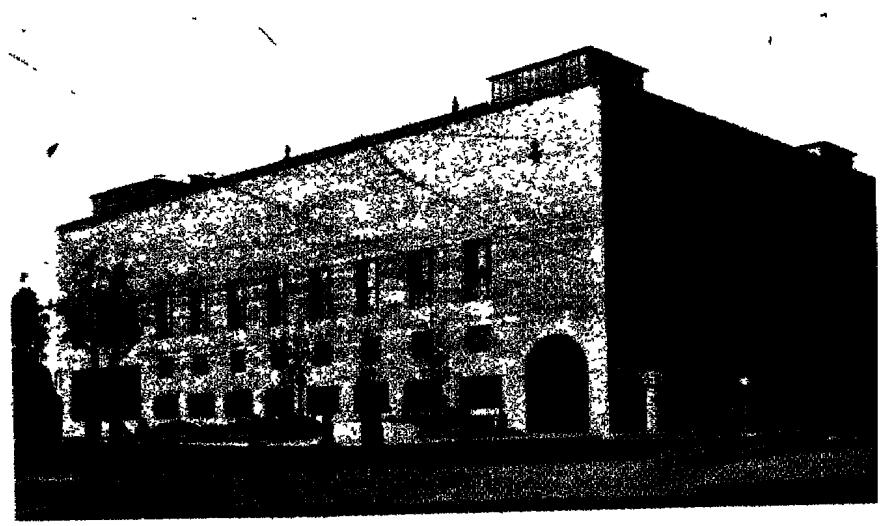


St. Anthony Church.

Examples of SWISS REINFORCED CONCRETE ARCHITECTURE; THESE ARE AT BASLE, SWITZERLAND



Covered Market



New Fine Arts Museum.

ALTUS DAM REBUILT AS GRAVITY STRUCTURE WITH MASONRY FACING USED AS FORMS

WAYNE S. BYRNE

Assistant Construction Engineer, Bureau of Reclamation, Altus, Okla.

CONTENTS IN BRIEF—*Formerly a slab-and-buttress type structure, Altus Dam in Oklahoma has been replaced by a much higher gravity dam with a stone masonry facing. This stonework was laid up one course at a time and was used as a form for the concrete, thus limiting the height of each pour to about 20 in. The narrow width of the structure near the top resulted in a varied procedure in placing the mass concrete, which consisted of a lean mix with the cement content reduced about 30 per cent. by addition of an inert pumicite.*

REPLACEMENT of Altus Dam in southwestern Oklahoma with a much higher structure required, under war conditions, the solution of problems not ordinarily encountered on this type of construction. The work was complicated further by a decision after construction was begun to build a concrete dam with stone masonry facing rather than an all-masonry structure

Built as the first project carried out in Oklahoma by the Bureau of Reclamation, the new structure spans the North Fork of Red River 18 miles from Altus, Okla., and immediately downstream from a low buttress dam built in 1927 as a municipal project to supply water for the city of Altus. It provides 152,000 acre-ft. of storage for irrigation of 60,000 acres of semi-arid land, for flood control and to supply water to the town of Altus. The original project was notable as the first and possibly the only Ambursen type dam to be built on a curve

New construction was begun early in 1941 by the Bureau of Reclamation with participation by the Works Projects Administration, the WPA planning to supply unskilled labour and small tools and the bureau to furnish some skilled labour and all engineering and supervision. Since liquidation of the WPA in 1942 the bureau has performed all construction work

In August, 1942 just before the work on the dam was to start, the War Production Board granted the priorities needed to complete the construction exclusive of irrigation features In October of the same year the WPB halted all work, but in December the stop order was modified to allow construction to continue to sufficient height to replenish a seriously depleted water-supply for the city of Altus and adjacent army installations, this work being completed in October, 1943 Later, or in April, 1944, the WPB, upon recommendation by the War Food Administration, granted authority for completion of the dam and distribution facilities to irrigate 20,000 acres by July, 1945.

Because of the long length of the new structure in relation to its height and the limited space for foundations, a straight gravity dam with a crest length of 1,104 ft and a maximum height of 97 ft. was found most feasible. To best fit the site, the central section of the structure was laid out on a radius of 487 ft. and the left abutment was designed to end as a reverse curve of 50-ft radius extending into a tangent section about 350 ft in length

Masonry structure first planned

Originally, the dam was designed as a granite masonry structure throughout This relatively outmoded type of construction was proposed in order to utilize the purported ample supply of WPA labour experienced in cutting and laying stone, and to permit a considerable saving of cement and lumber, when compared with an all-concrete dam However, the labour supply proved inadequate and subsequently the masonry was eliminated except as a 20-in thick granite facing for all upstream and downstream surfaces of the dam. An exception was construction of the downstream surface of the spillway section of concrete to give the proper hydraulic qualities.

The spillway, which has a total capacity of 60,000 cfs. with the upper pool level at El. 1564, is divided by a training wall into an uncontrolled section consist-



Fig. 1. Except for downstream face of spillway section, the dam is faced with masonry, encasing lean concrete.

ing of two 55-ft. openings with a crest at El. 1559 and a controlled spillway consisting of nine 21-ft openings with their sill at El. 1547. Eventually 15 × 21-ft. radial gates will be installed to provide storage up to El. 1559 by controlling the flow over this portion of the dam

Built in 20-in. lifts

Initially, only four contraction joints were provided along the dam, one at each end of the spillway section and one on each side of a canal outlet block. Because of the construction shutdown, provision was made for an additional contraction joint at each end of the controlled spillway from El 1523 to El 1547, or the crest of this section. Each joint is filled with asphalt in which steam pipes are embedded to permit the seals to be reliquified at proper intervals.

This wide space between contraction joints in combination with masonry facing called for more than ordinary care in construction to minimize cracking Consequently, the dam was built in 20-in. lifts with the day shift generally placing the masonry and the night crews the concrete. A modified cement and a lean mix in which about 30 per cent of the cement was replaced by an inert pumicite was used for all concrete in the masonry-faced sections In the spillway section where the concrete was to be exposed, a minimum of 3 ft of rich concrete (one bbl. of cement per cu. yd.) was placed against the forms

The use of pumicite to replace an equal volume of cement in the mass concrete had a twofold purpose. Primarily, the reduced cement content was intended to substantially aid in reducing the temperature rise of the mass concrete and thereby minimize cracking without the expense of artificial cooling. Second, since the major portion of the concrete was to be placed against granite masonry facing and consequently not exposed to weathering, the lean concrete would represent a considerable saving in cement.

Since 83 per cent. of the pumicite would pass a 325 mesh screen there was little difference in the workability of the "rich" and "lean" concrete, the latter developing an average compressive strength of 3,100 psi. at 28 days. The cost of the pumicite per cubic yard of concrete was about $0.25 which replaced an amount of cement costing $0.65, giving a clear saving of $0.40 on each cubic yard of lean concrete placed. No difficulties were experienced in handling the pumicite and no special apparatus was necessary.

Insofar as possible, concrete placing was confined to the winter months to eliminate the need for artificial cooling Added control of temperature rise in the mass concrete was obtained by continuous spraying of each lift for at least 72 hr

Old dam serves as cofferdam

The limited foundation area made it necessary to place the upstream face of the new dam within the area occupied by part of the old dam. Although this necessitated removal of the entire downstream apron and most of the supporting buttresses, it still was possible to use the existing structure as a cofferdam. To insure the stability of the old dam while it was acting as a cofferdam, the buttresses could not be cut back far enough to clear the new structure. Consequently, it was necessary to build the lower part of the face of the new structure around the buttresses, embedding them to a depth of about 6 ft

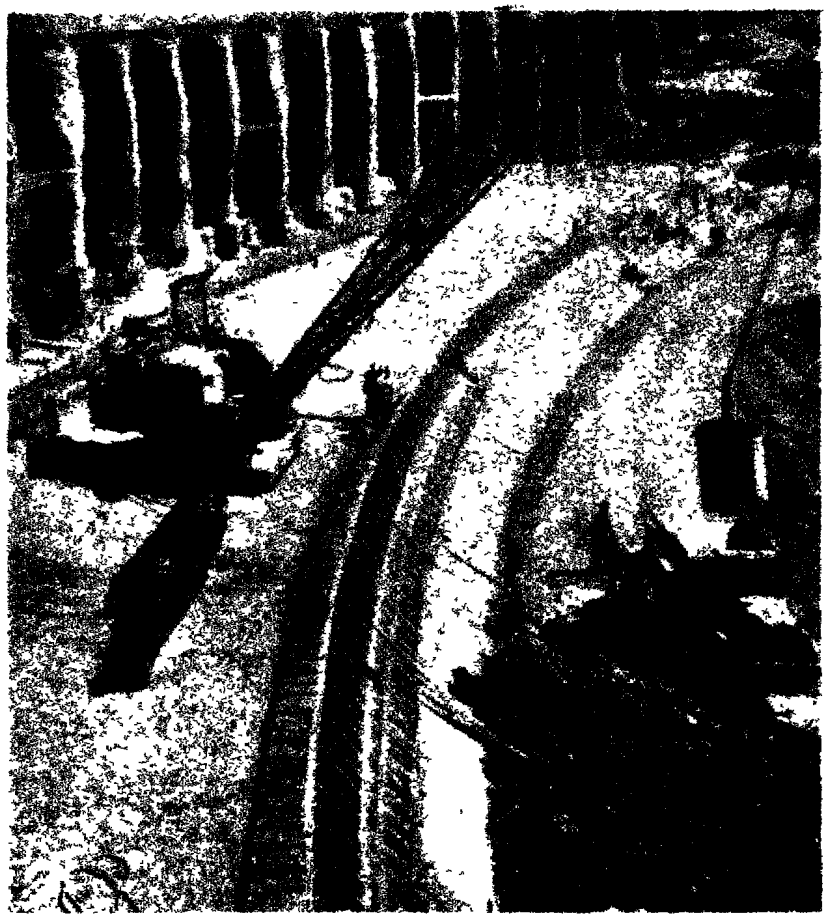


Fig 2 Concrete for apron and base of spillway section of dam was delivered by truck in 2-yd. buckets to a crawler crane operating on concrete placed several days previously. Old dam, which was used as cofferdam, is shown beyond crane.

Demolition of the old concrete created a problem. Drilling and blasting the old apron was too expensive, yet the sloping surface of the apron defied the use of a steel drop hammer and threatened to damage the boom of the crane handling the hammer Finally, a granite stone worn to a pear-shape from use as a drag on rip-rap slopes was tried. The abrasive granite prevented sliding and shattered the apron and the buttresses so completely that much of the apron reinforcing was salvaged for use in the new dam

Numerous "pot-holes" full of river silt made the foundation clean-up difficult. Hand labour was the only satisfactory means of exposing the rock surface, but in spite of the scour from years of river flow, the granite foundation rock was extremely sound and required little excavation. Roughening of the smooth, water-worn surface with air-tools proved to be the most difficult work in preparing the foundation rock to permit concrete placing to start.

River diversion was through a channel excavated in the left abutment for the canal outlet works and wasteway. A portion of this channel is the site of the canal outlet stilling basin and the remainder will be used as a wasteway An unusual feature of this work was that of providing support for the 24-in wood-stave pipeline of the Altus water-supply system. The pipeline crossed the channel at an angle that required suspension of 182 ft. of the pipe without benefit of piers.

Prior to excavating beneath the pipe, timber bents were constructed at either side of the channel, and cables parallel to the pipe were hung from them. Next, timber supports were forced under the pipe at 6-ft. intervals, the timbers subsequently being attached to the suspension cables by means of pre-cut suspenders and cable clamps When all were so attached the suspension cables were tightened uniformly by turnbuckles to support the pipe.

Excavation beneath the pipe was completed with a bulldozer and drag-line.

Central mixing plant used

Volume of new concrete totalled only 60,000 cu. yd., which limited the

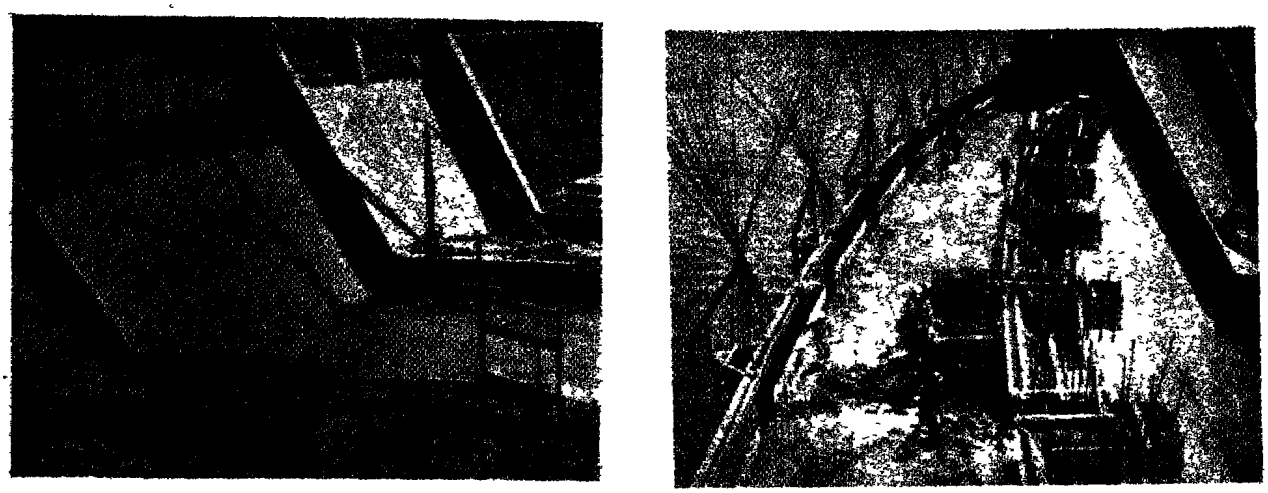


Fig. 3. Last concrete required for controlled section of spillway (left) was delivered with a bucket handled by a stiffleg derrick. Right: The concrete was dumped into an end-dump truck for distribution to desired point along dam. Because pours were of low height no segregation occurred.

type of mixing and placing equipment. All concrete and mortar was mixed in a central plant by means of 2-cu. yd. tilting mixers fed from a manually operated batching plant by a 135-ft conveyor belt fed from hoppers.

The pumicite, a fine, white powdery material consisting almost entirely of pure silica, was mined and loaded with a bulldozer and hauled in trucks about 20 miles to the site of the dam At the batching plant it was screened to remove lumps and weighed prior to being placed on the belt with the cement and coarse and fine aggregates, the entire batch being dumped into a charging hopper over the mixer The aggregate stockpiles were located in a circle with a guy derrick in the center, permitting the derrick to place material in any bin with slight variations of the boom position

Mixed concrete was dumped into a receiving hopper and then into 2-cu yd buckets and hauled to the dam in 13-cu yd bottom-dump trucks This type of truck proved the fastest and most manœuvrable of all the vehicles available

As the major portion of the concrete was required for the spillway section, a system of ramps was built to permit trucks to deliver concrete for construction of a paved strip along the downstream toe of the dam sufficiently wide to accommodate a crawler crane equipped with a 90-ft boom Working on the paved area, the crane was able to place concrete for the entire spillway apron and the foundation of the dam proper

When the spillway apron had been completed an earthfill ramp was placed across it, and the concrete and stone was placed by crawler cranes operating on mats along the top surface of the dam proper. This method became impracticable when the top width of the dam was reduced to less than 15 ft., thus making a third method necessary to complete the work

For this latter method, a 4-cu yd dump truck, outfitted with a 3,000-lb derrick and with the outer rear wheels removed to enable the vehicle to operate between the upstream and downstream stonework, was used to place the stone facing (Fig 4) During this stage, concrete was delivered with a 2-cu. yd. dump truck equipped with special single rear wheels and oversize tires A crane working at one end of the dam lifted 2-cu yd. buckets from below and dumped the concrete into the bed of the truck, which transported it to the required location Since all mass concrete was placed in 20-in lifts with a slump of 2-in. or less, no segregation occurred when the concrete was dumped from the truck.

When imbedded steel for the piers made the uncontrolled section of the spillway too narrow to permit passage of the truck, a temporary construction bridge was built above the spillway for delivery of the concrete An old 3-cu yd. concrete bucket was converted into a hopper and placed at one end of the bridge, where it could be filled by the crane. From the hopper



Fig. 4. After a thorough cleaning with a high-pressure water jet, the granite for masonry facing was set in layer of mortar with a rubber tired truck equipped with a 3,000-lb. capacity derrick. When the mortar had taken its initial set, the joints were "wiped" with burlap to produce rough effect blending with the stone surface. All joints were broken and headers were placed at a maximum spacing of 5 ft.

KRÄZERN BRIDGE, SWITZERLAND

THE reinforced concrete bridge illustrated replaces a stone viaduct over the Sitter river, near St. Gall, Switzerland. In 1936 a competition was held and the contract was awarded to a group of firms, namely Messrs. H. Ruesch, J. Muller & Co., Sigrist, Merz & Co., and Zublin (the last mentioned firm was responsible for the shuttering) The design and calculations were made by two consulting engineers, namely M. Ch. Chopard of Zurich and M. A. Brunner of St. Gall.

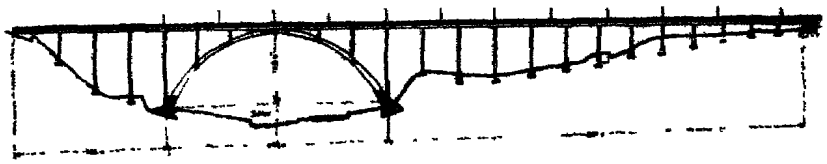


Fig. 1.—Longitudinal Section.

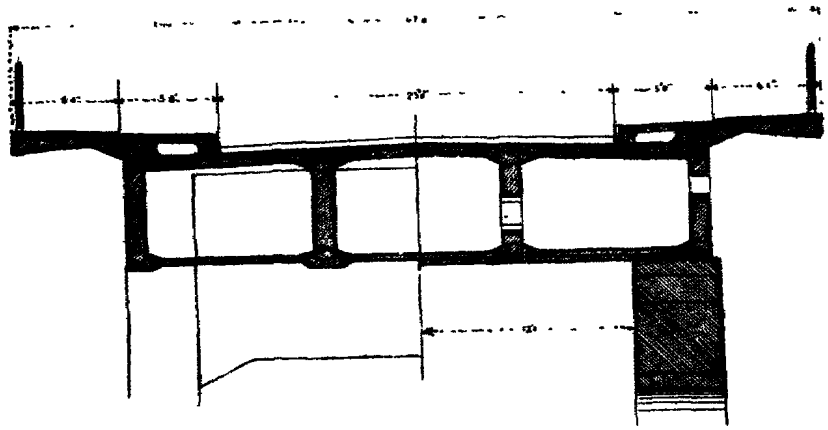


Fig. 2.—Cross Section.



Fig. 3.—Kräzern Bridge, Switzerland

The arch across the river has a span of 442 ft. and a rise of 148 ft., giving a rise/span ratio of 0.335. The arrangement adopted is a system of two fixed-end arch ribs, whose cross section has a slightly trapezoidal shape. The ribs are 7 ft. 8 in. deep by 5 ft. 2 in. at the crown and 14 ft. 2 in. deep by 7 ft. 6 in. wide at the springings. Winbracing consists of eight transverse beams connecting the ribs. The deck, which provides for a 25-ft. roadway and two cycle paths each 5 ft. 6 in. wide, is carried on four longitudinal beams. There are also two cantilevered footpaths each 6 ft. 4 in wide.

The approach viaducts consist of continuous girder spans carried on reinforced concrete portal frames. The bridge has a total length of 1,609 ft.

The concrete was vibrated, and tests on 8-in. cubes from concrete being placed in the bridge showed a crushing strength of 6,400 lb per square inch at 28 days. The reinforcement bars in the arch are chrome steel having a tensile strength of 82,500 lb per square inch and a yield point of 55,400 lb. per square inch. These bars were designed for a working stress of 22,700 to 26,000 lb. per square inch

The falsework consisted of an asymmetrical lattice timber arch supported on two fans; the centering had a settlement of only a 1½ in under the load of the two main ribs The materials used in the contract were as follows: Concrete (reinforced), 10,250 cu yd.; concrete (mass), 5,800 cu yd.; reinforcement (mild steel) 630 tons, reinforcement (high tensile steel), 450 tons

The bridge was subjected to a test load before being opened to traffic. During the test eight 10-ton trucks were placed to produce the maximum stresses at various sections. The maximum deflection at the crown did not exceed 1.05 mm. This agreed with the engineers' calculations based on a theory, confirmed by tests on models, in which the arch ribs and decking act as a monolithic structure. The bridge was opened to traffic in September 1941.—(*With acknowledgments to "Concrete and Constructional Engineering."*)

(*Continued from page* 209)

the concrete was transported in buggies to trap doors spaced at 10-ft intervals, below which were "elephant trunks" reaching to the surface of the dam.

Still more inaccessible were the controlled spillway and the eight spillway piers. To place concrete in these parts of the structure it was necessary to backfill in front of the uncontrolled spillway apron with earth to the top of the intermediate training wall A crawler crane, operating on this fill and delivering concrete to the dump truck for distribution in the manner previously described, permitted construction to continue up to El. 1537, above which a stiffleg derrick with a 120-ft. boom was substituted for the crane (Fig. 3).

The work of completing the Altus Dam and a main canal 4.4 mi. long is being performed by force account. Morrison-Knudsen Co., Boise, Idaho, is constructing a 2,000 ft. siphon under the north fork of Red River.

The Altus project is under the supervision of the Bureau of Reclamation. During most of the construction period H. W. Bashore, was commissioner, Wesley R. Nelson, regional director, S. O. Harper, chief engineer and J. L. Savage, chief designing engineer. H. E. Robbins was construction engineer in direct charge. He was assisted by the writer as assistant construction engineer and by P. J. McGovern, construction superintendent; R. A. McCallum, assistant field engineer; and K. L. Powers, materials engineer.—(*With acknowledgments to "Engineering News-Record."*)

GRAPHICAL DATA FOR CONCRETE COLUMNS

By W. SCOTT WILSON

THE accompanying graphical data show at a glance the safe load in tons carried by columns of various dimensions for a given eccentricity " e " and steel percentage " p," based on a maximum skin stress of 750 lb. per sq. in. Referring to Fig. 1, for instance, a 14 in. × 14 in. column reinforced with 2 per cent. steel supports a load of 48 tons when the eccentricity " e " is 2 in Again the same column supports a load of 20 tons when the eccentricity " e " is increased to 7 in, the maximum skin stress remaining at 750 lb. per sq. in. If it is desired to use a higher safe stress, say 850 lb. per sq. in., then the safe load read from the curves may be increased in the ratio 850/750.

In practice it sometimes happens that the eccentricity " e " is known as, for example, a load transmitted to a column through the medium of a bracket, but, in the majority of cases, the eccentricity " e " is involved with a bending moment " M " and a load " W ". The moment, therefore, has first to be determined, the eccentricity following from the relation W × e = M.

This condition obtains in the usual type of framed structure shown on Fig 4. When the main beams deflect under load, their ends, although restrained by the stiffness of the columns, do not remain horizontal. This means that the columns above and below the beams turn through the same angle—the nature of the distortion being indicated on Fig. 4—and consequently a bending moment M is induced in the columns.

The magnitude of this bending moment depends upon the relative stiffness of the beam and column. If the column is very stiff compared to the beam the end of the beam will tend to remain horizontal under load and the bending moment in the beam where it enters the column will approximate to W × L/12.

If the column is very slender compared to the beam it will offer very little restraint to the beam end and, therefore, the bending moment will approximate to zero. All practical cases lie somewhere between these limits.

To illustrate the points involved consider the main beams on Fig. 4, which support two point loads of 23 tons acting at third points and producing a bending moment on the beam equal to 129 ft tons, or 3,470,000 in. lb. The beam section is 24 in. × 12 in reinforced with 8 bars 1⅛ in. diam., and the compression at the extreme fibre of the 4 in. slab is 770 lb. per sq in.

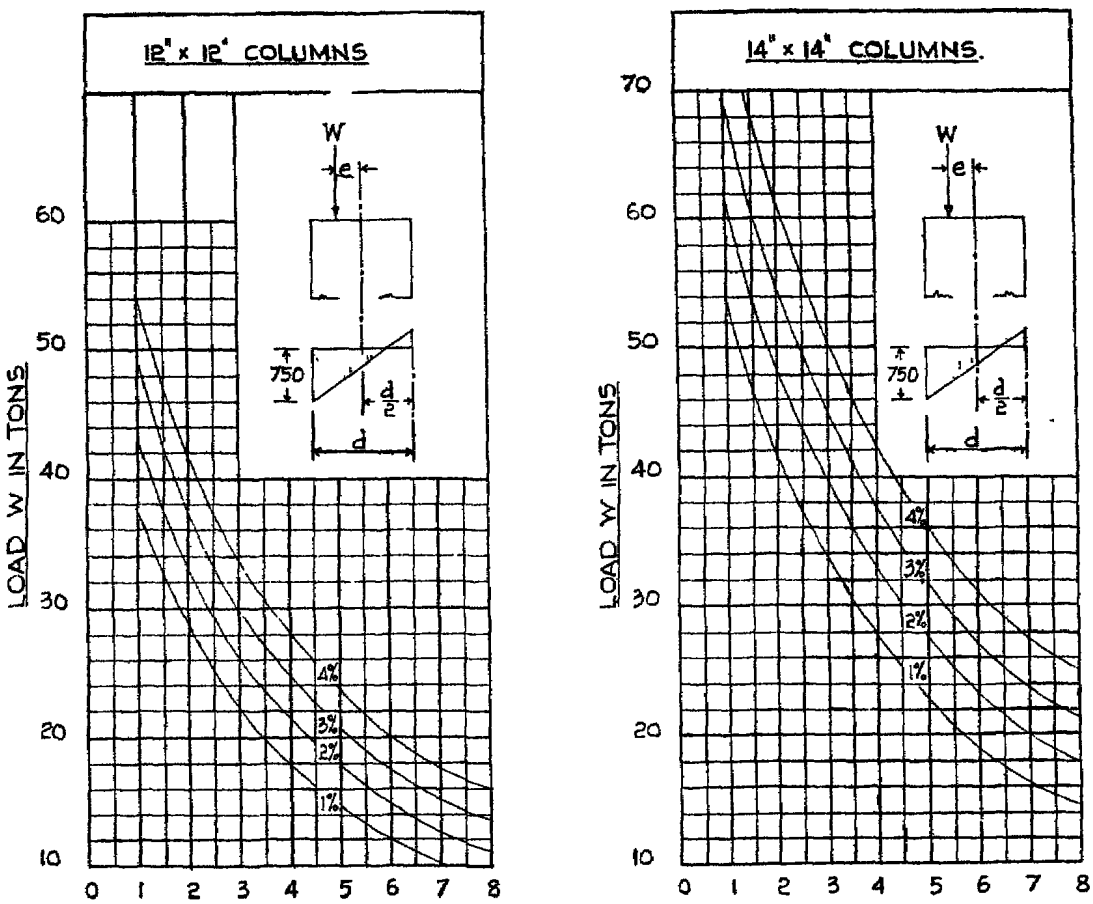


FIG 1

then, ratio of stresses t/c = 18,000/770 = 23·4

and value of n = 0·39

distance of neutral axis from top of slab nd = 0·39 × 23 = 9 in.

Inertia of beam Ib = M × y/stress
= 3,470,000 × 9/770
= 40,500 ins⁴

Stiffness of beam Sb = inertia lb./length of beam
= 40,500/24 × 12
= 140

The load W on the columns under second floor is 40 tons, and the load W on the columns under first floor is 80 tons. Selecting sections 14 in. × 14 in and 16 in. × 16 in. with 2 per cent. and 4 per cent. steel respectively, we have :—

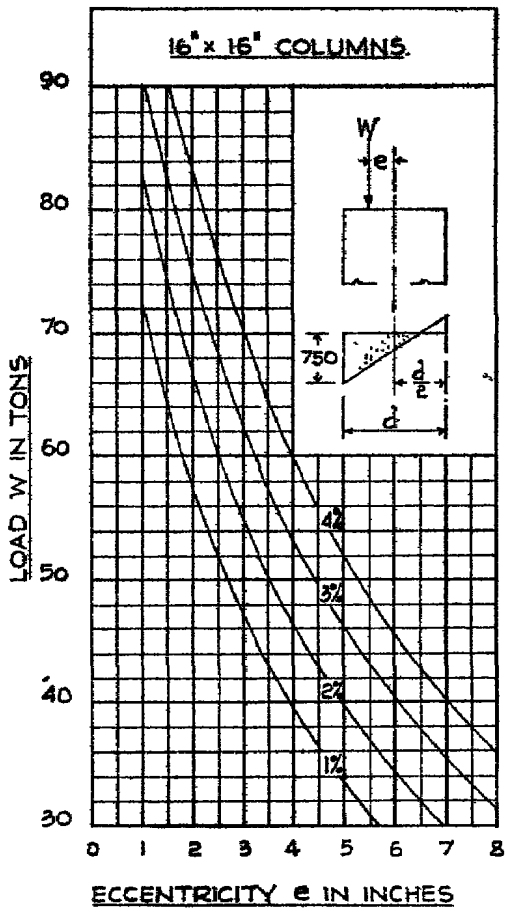
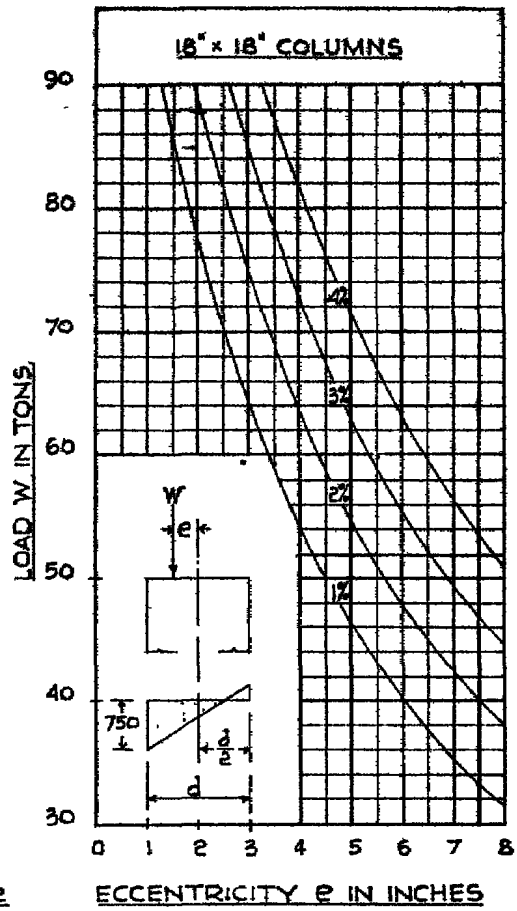

16" x 16" COLUMNS
LOAD W IN TONS
90
80
70
60
50
40
30
0 1 2 3 4 5 6 7 8
ECCENTRICITY e IN INCHES
W
e
750
d/2
d
4%
3%
2%
1%

FIG 2


18" x 18" COLUMNS
LOAD W IN TONS
90
80
70
60
50
40
30
0 1 2 3 4 5 6 7 8
ECCENTRICITY e IN INCHES
W
e
750
d/2
d
4%
3%
2%
1%

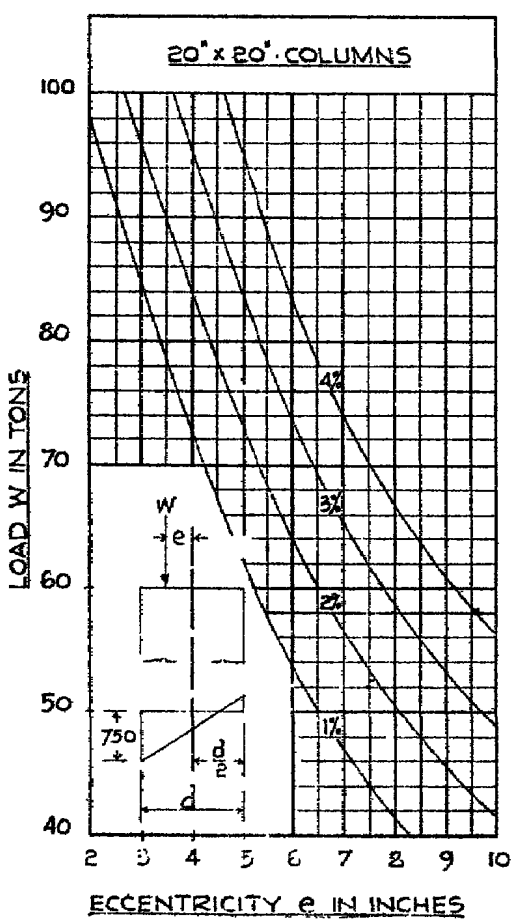
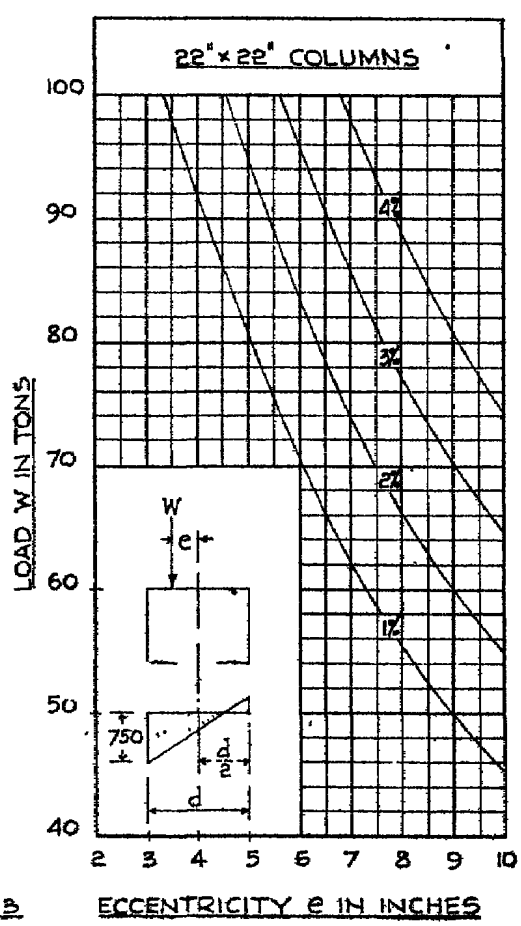

20" x 20" COLUMNS
LOAD W IN TONS
100
90
80
70
60
50
40
2 3 4 5 6 7 8 9 10
ECCENTRICITY e IN INCHES
W
e
750
d/2
d
4%
3%
2%
1%

FIG 3


22" x 22" COLUMNS
LOAD W IN TONS
100
90
80
70
60
50
40
2 3 4 5 6 7 8 9 10
ECCENTRICITY e IN INCHES
W
e
750
d/2
d
4%
3%
2%
1%

Inertia of upper columns :

I_u = 4,800 ins.^{4}

and stiffness S_u = inertia Iu/height h of column
= 4,800/14 × 12
= 28·3

Inertia of lower columns :

I_L = 10,700 ins.^{4}

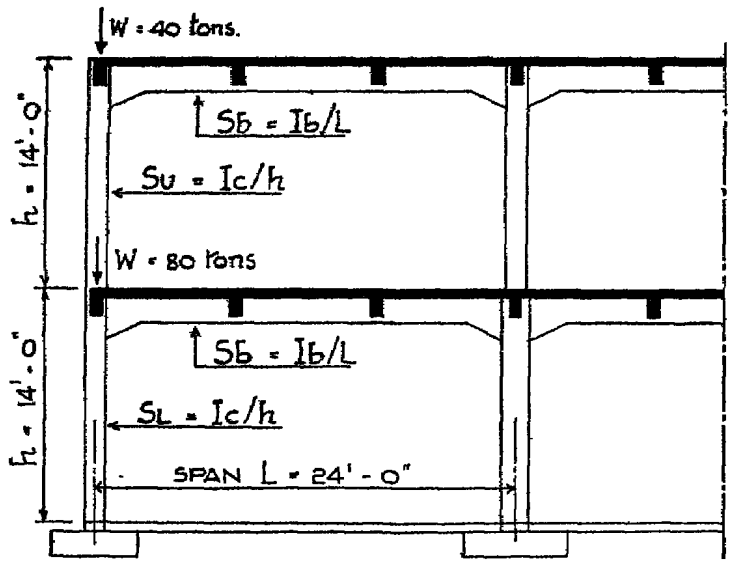
and stiffness S_L = 10,700/14 × 12.
= 63·7.

Columns under first floor :

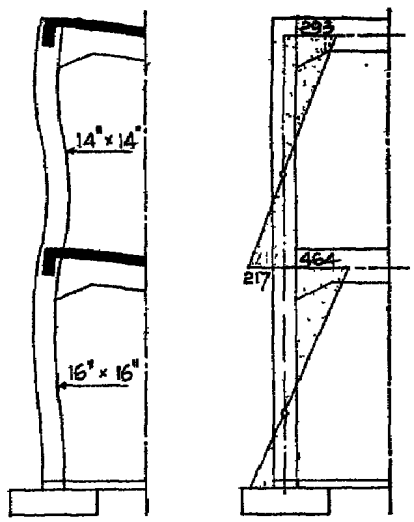
moment factor $= \frac{Su + Sl}{Su + Sl + Sb}$
$= \frac{28·3 + 63·7}{28·3 + 63·7 + 140}$
= 0·4

Negative moment in main beam at junction with colun

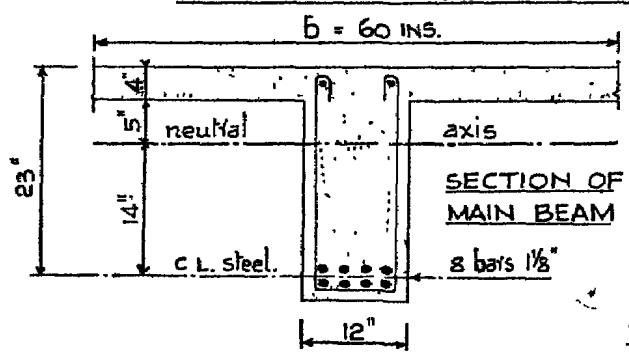
$-Mb$ = 0·4 × W × L/9
= — 0 044 W × L
= 0 004 × 24 × 2,240 × 24 × 12
= 681,000 in lb.



SECTION OF FRAMED STRUCTURE



DISTORTION AND MOMENTS IN EXTERNAL COLUMNS.



INERTIA OF BEAM :-

I_b = 3,470,000 × 9/770
= 40,500 ins^4

OR $I_b = M \times y / stress$

FIG. 4

Columns under second floor :

Moment factor $= \frac{Su}{Su + Sb}$
$= \frac{28·3}{28·3 + 140}$
= 0·17

Negative moment in main beam at junction with column :—

$-M_b$ = 0 17 × W × L/9
= — 0·019 W × L
= 0·019 × 24 × 2,240 × 24 × 12
= — 293,000 in. lb.

The moment in the column is equal to the moment in the beam but may be reduced on account of the haunch :—

Moment in column = 293,000 × 6/8·5
= 207,000 in. lb

load on column W = 40 tons = 90,000 lb.

eccentricity e = M/W = 207/90
= 2·3 in.

Consulting Fig. 1 we see that columns 14 in. × 14 in reinforced with 2 per cent. steel (4 bars 1¼ in diam.) support a load of 45 tons with an eccentricity of 2·3 in., and are therefore suitable.

The algebraic sum of the moments around the junction must equal zero, hence we can write —

Moment at base of upper 14 in. × 14 in. columns :—

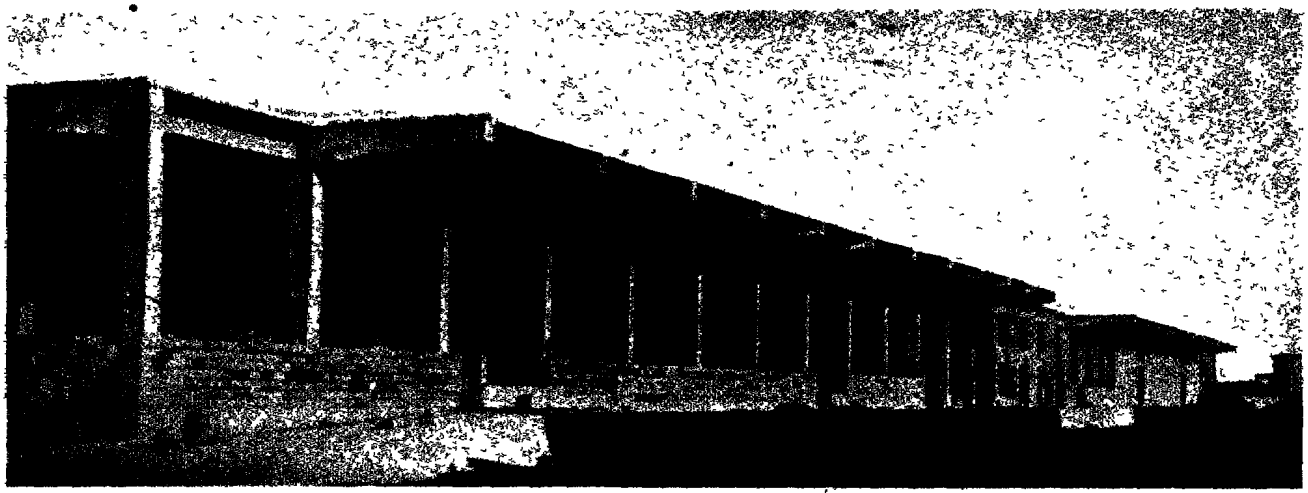
$Mu = \frac{Su}{Su + Sl + Sb}$
$= \frac{28·3}{28·3 + 63·7 + 140}$
= 0·125 × W × L/9
= 0·014 × W × L
= 0 014 × 24 × 2,240 × 24 × 12
= 217,000 in lbs

Moment at top of lower 16 in × 16 in columns —

$Ml = \frac{Sl}{Sl + Su + Sb}$
$= \frac{63·7}{63·7 + 28·3 + 140}$
= 0·275 × W × L/9
= 0·03 W × L
= 0·03 × 24 × 2,240 × 24 × 12
= 464,000 in. lb.

that is —

$M_u + M_L - M_b$ = zero



All Photos Official U. S. Navy

Fig 1 Framing of one warehouse of precast structural members was completed in 40 days and a second in only 18 days.

PRECAST STRUCTURAL MEMBERS FACILITATE SPEEDY ERECTION OF RIGID-FRAME BUILDINGS

W. MACK ANGAS

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Contents in Brief—*Precast concrete roof slabs and hollow-box rigid frames made rapid erection of two, one-storey warehouses at Mechanicsburg, Pa , possible at relatively low cost These members, made with high early strength cement and drained of excess water by the vacuum process, were lifted from the forms within 24 hr. after casting by suction pads composed of light steel frames faced with plywood and bordered with soft rubber gaskets. Ingenious assembly methods speeded erection.*

NOVEL use was made in the construction of two, large single-storey warehouses at the Naval Supply Depot, Mechanicsburg, Pa , of thin channel-shaped concrete sections Bolted together, these precast members formed hollow-box girders and columns, which were integrated into rigid frames upon erection by welding of the reinforcing and concreting of the joints With no precedent to go by, the contractor devised some interesting and effective means for precasting the thin sections, handling the green concrete and erecting the structure

The innovations in design of the Mechanicsburg warehouses and the unique methods developed by the contractors for the construction of the buildings, promise to produce a type of fireproof structure that can be built about as quickly and cheaply, in quantity, as existing types of semi-permanent buildings that are, to say the least, extremely vulnerable to damage by fire Features of the design of these warehouses and methods used in their construction might be applied profitably to many types of post-war construction

The concrete foundations and floors of the buildings are of the conventional type and were constructed by conventional methods, as were the hollow concrete-block, exterior walls (Fig 1) The columns and girders supporting the roof are, however, not of ordinary construction They were built of precast reinforced-concrete members assembled by a combination of bolting, welding and concreting to form rigid frames of extremely light hollow members (Fig. 3) The quantity of concrete poured in place to connect the precast members was very small

The concrete roof slab consist of 100-sq ft precast panels, which span the 20-ft space between the rigid frames, with no joists or purlins other than the girders and beams cast integrally with the panels (Fig 2)

Exterior columns, T caps of the interior columns, and girders of the rigid frames were cast in two pieces that were bolted together to form hollow box members, which were stiffened by interior diaphragms Interiorcolumns and horizontal struts connecting the bents were cast in a single piece, the hollow cores of these members being formed by a treated cardboard tube Left in place, the tube was removed only at the upper and lower ends of the columns to permit the exposed concrete to be cleaned by sandblasting so as to provide a good bonding surface for the cast-in-place concrete used to connect the precast members

All precast members were poured in an outdoor casting yard, using forms of hard-surfaced and smooth-trowelled concrete, with sides of wood hinged

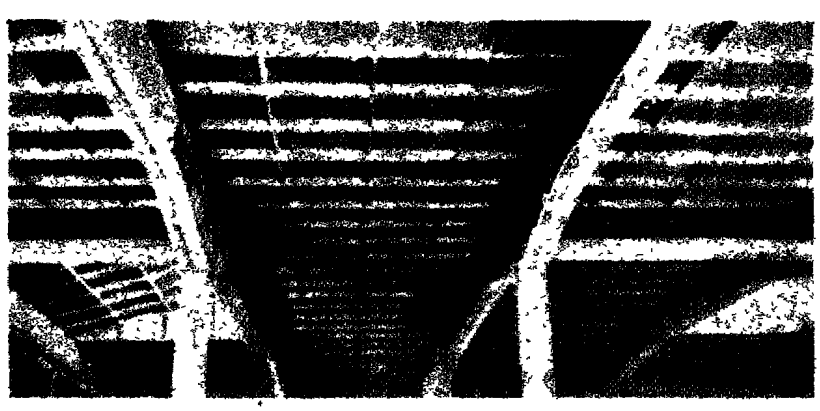
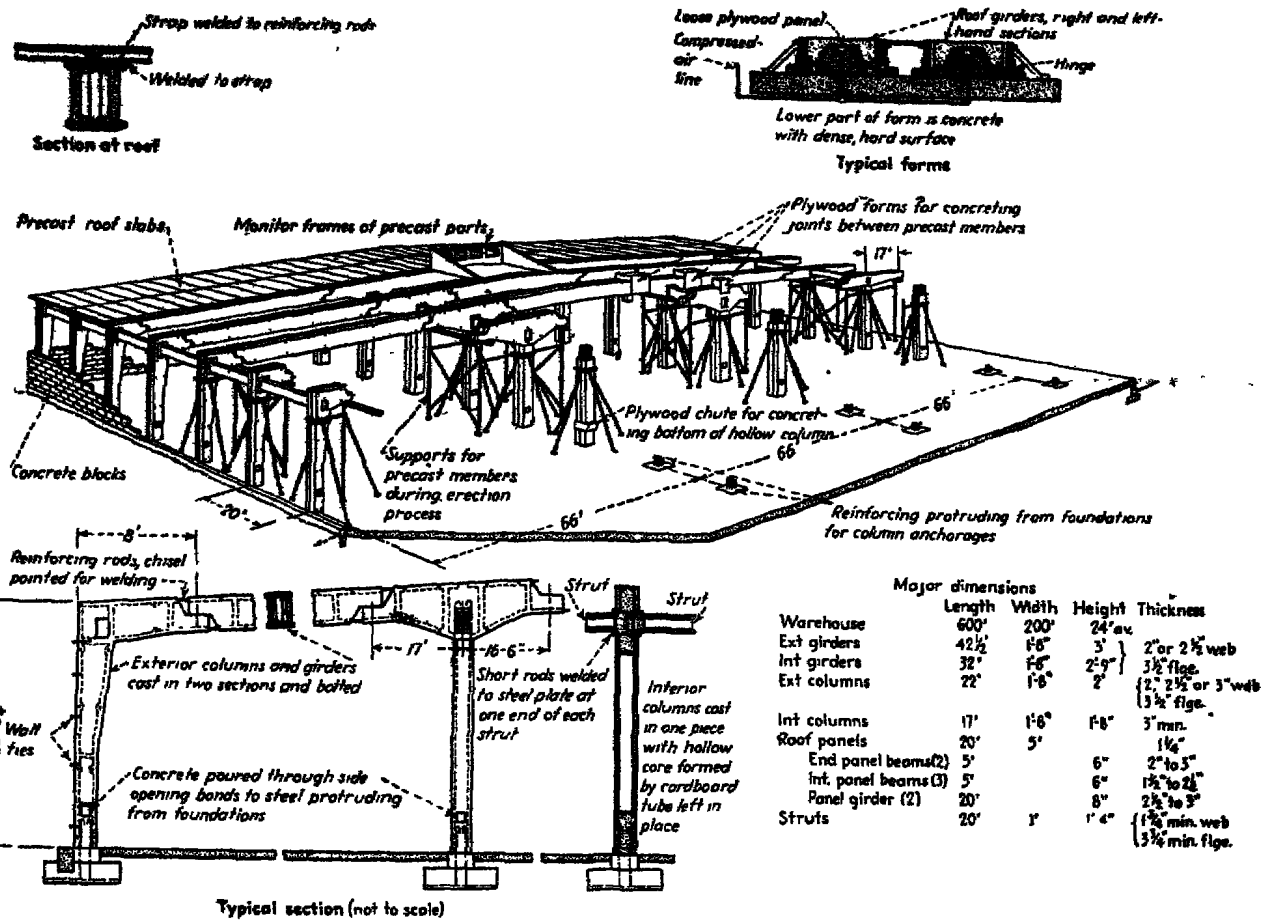


Fig. 2. Roof panels with exposed beams and girders that were precast integrally with slab 20 ft. between precast rigid frames of hollow box section.



Major dimensions	Length	Width	Height	Thickness
Warehouse	600'	200'	24' av.	
Ext girders	42½'	1'-6"	3'	2" or 2½" web
Int girders	32'	1'-6"	2'-9"	3½" flge.
Ext columns	22'	1'-8"	2'	2", 2½" or 3" web; 3½" flge.
Int columns	17'	1'-8"	1'-8"	3" min.
Roof panels	20'	5'		1¼"
End panel beams (2)	5'		6"	2" to 3"
Int. panel beams (3)	5'		6"	1½" to 2½"
Panel girder (2)	20'		8"	2½" to 3"
Struts	20'	1'	1'-4"	1½" min. web; 3¼" min. flge.

Fig. 3. Methods employed in assembly and erection of the precast warehouse framing are illustrated in this schematic drawing.

to promote rapid removal and reassembly Excess water was removed from all precast sections by the vacuum process to promote soundness, strength and rapid curing To make it possible to remove the members from the forms and transport them to the curing and storage yard within 24 hr. after pouring, high early strength cement was used in the mix, and suction pads, supporting the sections throughout their entire length, were ingeniously used for lifting (Fig. 4).

The suction pads consisted of light structural steel frames faced with plywood and surrounded with soft rubber gaskets. There were one or more pads for each type of precast section to be handled shaped to fit the upper surface of the member it lifted.

A crane handled the lifting pads To raise a member, the pad was placed

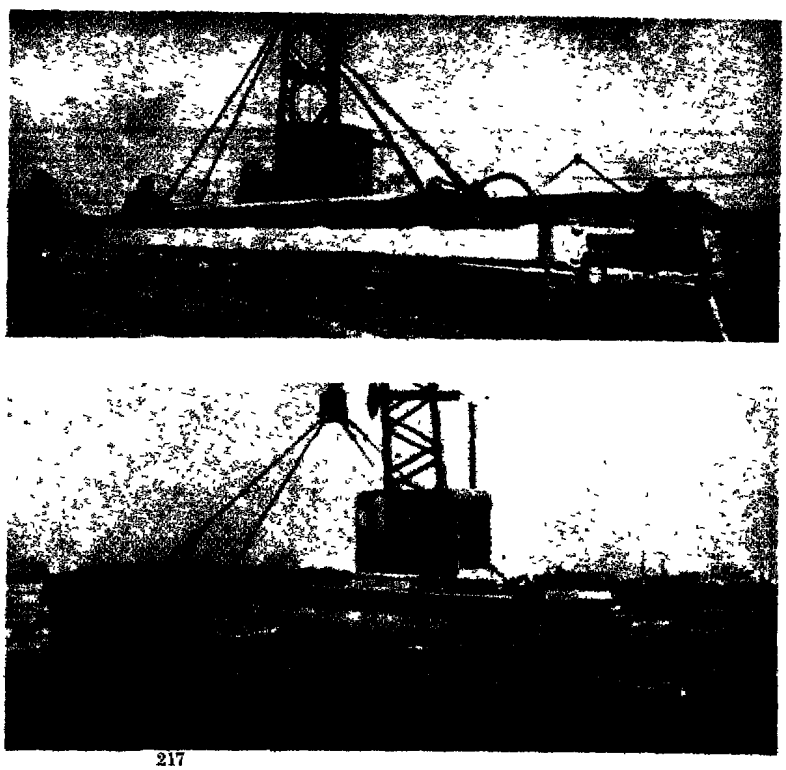


Fig. 4. Half of rigid-frame girder (top) was lifted from the form by a suction pad less than one day after casting and placed on its mate for bolting in storage yard. Roof slab (bottom) was similarly transported from forms to storage Curved steel bars protruding from ends of suction pad were planned to catch member if suction should fail.

on it, the vacuum hose connected to a pipe protruding from the back of the pad, and the air between the facing and concrete surface partially exhausted. The crane then lifted pad and member almost as easily and quickly as a lifting magnet handles steel or iron. A vacuum of about 12½ in of mercury was used, formed in a few seconds and easily maintained

Pipes connected to vacuum pumps, run through the casting, storage and assembly yards, served not only to remove excess water from newly cast members, but also to energize the ingenious suction pads. When the precast members were lifted from the forms, a little compressed air was admitted through a pipe leading to the upper surface of each form to break the seal between its greased concrete surface and the member to be lifted from it. This unique method of lifting and handling precast concrete members in the casting, curing and assembly yards did much to expedite construction and lower costs

In the actual erection of the buildings, all members were lifted by slings in the ordinary way. A crane set columns in place on concrete foundations from which reinforcing rods had been left protruding to provide anchorage (Fig. 3) These rods extended up into the hollow cores of the columns, which were temporarily braced and held vertical by steel struts, each of which was fitted with a turnbuckle to facilitate rapid and accurate plumbing of the columns The lower ends of the columns were then filled with concrete which, when it set around the reinforcing steel, anchored them securely.

The T-heads used on the interior columns and on the columns on the

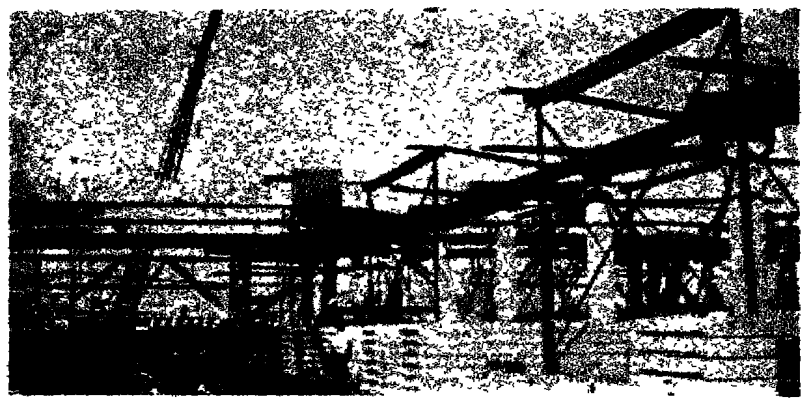


Fig. 5. Tubular steel scaffolding supported rigid frames during erection until concreted joint had hardened. Roof slabs were stored in warehouse area.

canopy sides of the warehouses were then similarly set and secured by pouring concrete into their central sections into which steel reinforcing rods extended from the upper ends of the columns Each column head pour also tied an end of one of the horizontal struts connecting the building frames, the other end of the truss being fixed by bolting

Precast box girders were then lifted into place, and reinforcing rods left protruding from them were welded to similar rods protruding from the concrete of the column caps After the rods were welded, concrete cast in plywood forms around the joints between members of a bent connected them into a continuous rigid frame This bestowed upon the bent the essential characteristics of a monolithic structure combined with extreme lightness and economy of both material and labour

The precast roof panels of the warehouses were fastened in place by welding steel inserts to steel straps that had been bolted to the faces of the roof girders, the inserts themselves having been welded to the main reinforcement of the panels before these were poured. The monitors on the roof were similarly built up of precast members. The laying of a built-up felt and asphalt roof membrane over the precast panels and the installation of lighting and heating systems completed the building.

The warehouses were designed by the Bureau of Yards and Docks of the Navy Department, the designer principally responsible for their unusual characteristics being A. Amirikian. Unusual methods used in their construction were developed by the contractors, Corbetta Construction Co of New York, N Y , and Vacuum Concrete, Inc. of Philadelphia, Pa.—(*With acknowledgments to "Engineering News-Record"*)

THE FIRST DEPUTATION OF GOVERNMENT HIGHWAY ENGINEERS FROM INDIA ON A VISIT TO ENGLAND.



On May the 15th, the First Deputation of Govt. Highway Engineers from India visited the Watford Works of Blaw-Knox Limited of London, England

Some 24 members of the deputation witnessed demonstrations on the Work's trial ground, of British built earth moving equipment and also the Blaw-Knox Concrete Spreading Machine. Following this demonstration, they inspected and observed the running of a range of British Built "REX" Concrete Mixers. Blawform steel shuttering was also on view and demonstrated.

During the tour of the workshops, the visitors witnessed the building of construction equipment, including earth moving scrapers, angledozers, rippers, sheepsfoot tamping rollers, roadforms, Rex Concrete Mixers, Blaw-Knox Concrete Pumps and other items of equipment. Great interest was shown in a mobile batching plant, one of two under construction for the British Air Ministry. These plants, which are mounted on pneumatic tyred wheels, are designed supply batched by weight materials, paving machines or concrete mixe capable of producing 34 cu. ft. of mix concrete per minute.

A PRESTRESSED CONCRETE FOOTBRIDGE

THIS footbridge over the tracks at the Bully-Grenay station of the French National Railways has two spans (*Fig.* 1) of 98 ft. 6 in and 101 ft 8 in. respectively. Each span comprises two prestressed concrete girders connected by a deck slab [*Fig.* 2(*b*)]. A condition was that the girders should be 6 ft. deep in order to provide a 5 ft. parapet to the footway. Otherwise a girder only 4 ft 3 in. deep would have been sufficient [*Fig* 2 (*c*)].

A comparison between the quantities of materials used for three alternative designs in ordinary reinforced concrete and prestressed reinforced concrete is given in *Table* 1, which shows that for the prestressed designs there is a saving in weight of steel of about 60 per cent compared with ordinary reinforced concrete

TABLE 1.

Design	(*a*)	(*b*)	(*c*)
Concrete (cu yd) ..	17	17	15
Mild steel (lb) ..	11,600	2,700	2,100
Hard steel (lb) .	—	1,900	1,900
Total steel (lb) ..	11,600	4,600	4,000
Shuttering (sq yd)	160	160	140

The reinforcement consists of prestressed cables, composed of hard-drawn steel wires each of $\frac{1}{8}$ in. diameter, having a tensile strength of 213,000 to 227,500 lb per square inch, and a yield stress of 170,000 to 213,000 lb. per square inch. Each wire was given a total tension of 3,960 lb (equivalent to 135,000 lb per square inch), and stretched between the anchorages by means of jacks Cables of the wires were placed in tubes of 1-in. diameter. After the wires had been stressed, the tubes were grouted under pressure.

A high-quality concrete with high strength at an early age was obtained by suitable grading of the aggregates and by intense vibration. The proportions were . fine sand (from 0 to $\frac{1}{8}$ in), 8 cu ft., coarse sand (from $\frac{1}{8}$ in. to 1 in.), 20 cu ft ; cement, 850 lb per cubic yard The water-cement ratio was 0 42. Tests on 8-in. cubes made from the concrete being placed, gave a crushing strength of 5,800 lb. per square inch at 90 days

The girders, which were precast, weigh about 34 tons each, and were provided with hoisting rings. After hardening, they were taken to the site on railway wagons, lifted by locomotive cranes and placed on the pier and abutments (*Fig.* 3). The time required for these operations was only 1½ hours. The deck between the prestressed concrete girders is in ordinary reinforced concrete, the shuttering for which was suspended from the girders.

The footbridge was completed in 1944 and was constructed by the etablissements Sainrapt & Brice, of Paris,—(*With acknowledgments to "Concrete and Constructional Engineering."*)

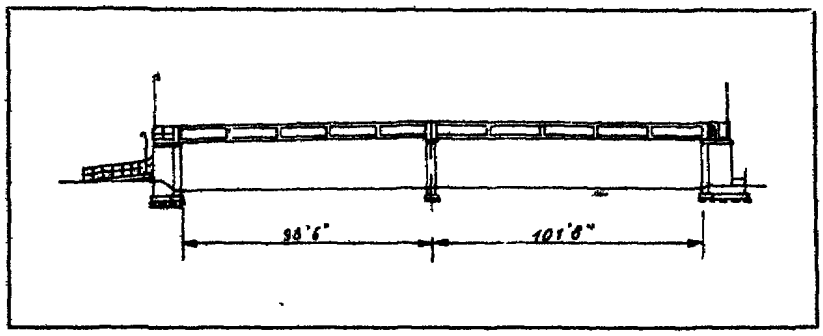


Fig. 1.

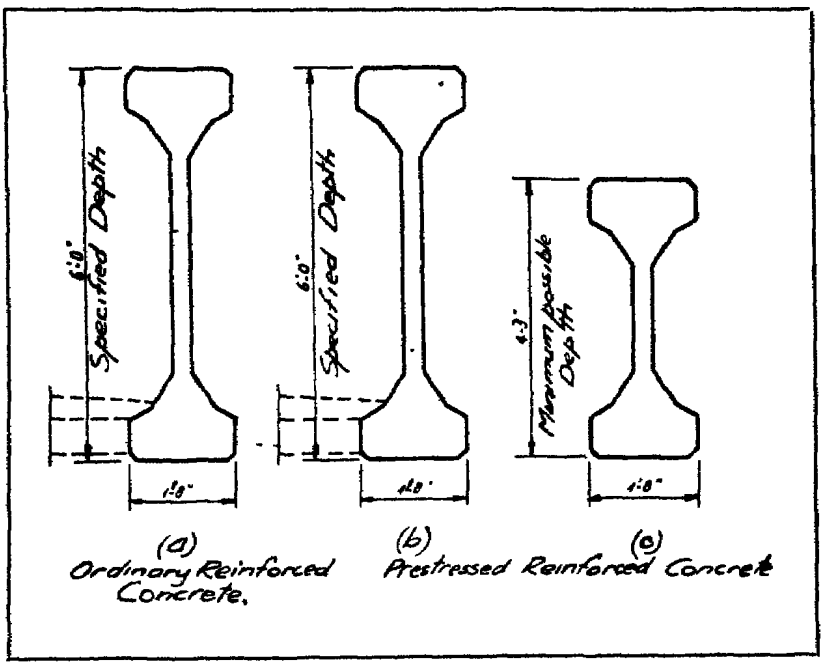
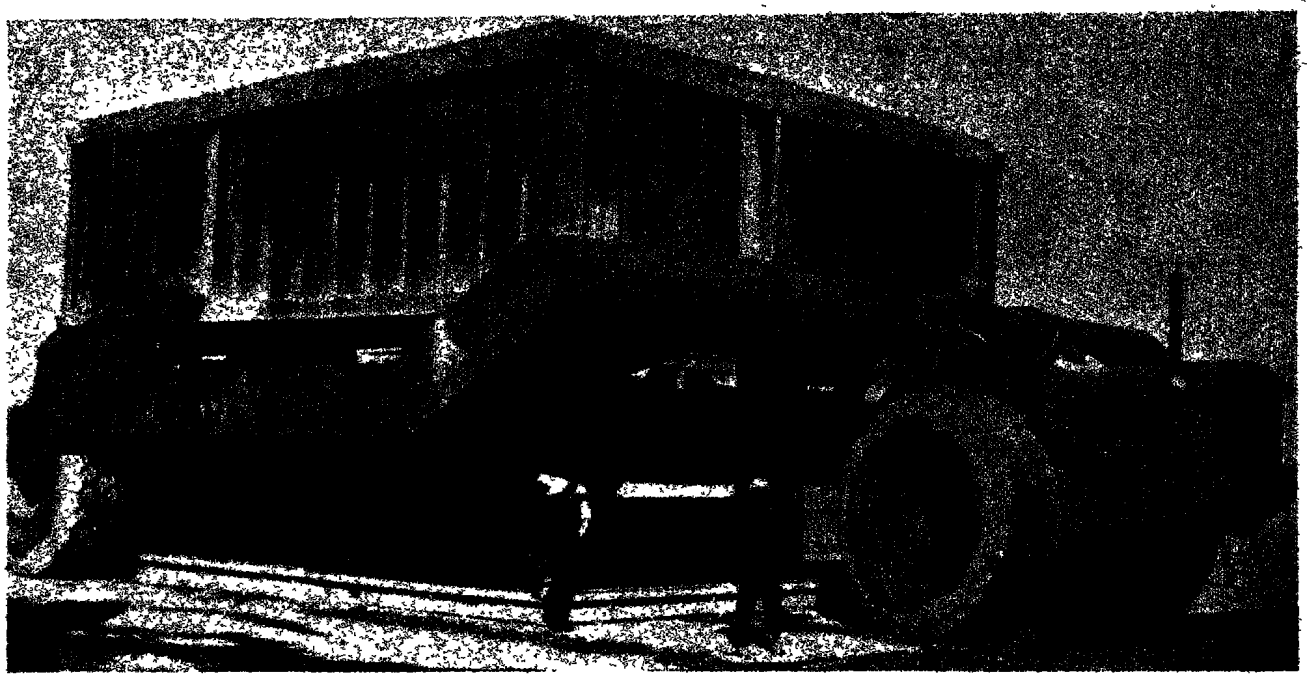


Fig. 2.



Fig. 3.



HOUSE HATCHING MACHINE drops outside steel forms over inner form assembly, in which reinforcing door and window frames have been placed, at central casting yard. LeTourneau machine, called Tournalayer, consists of two-wheel U-frame trailer towed by two-wheel Tournapull tractor.

"BUNGALOW" BIDDY HATCHES CONCRETE HOUSES

MONOLITHIC Concrete Houses precast at a central yard and moved to final location by a gigantic tractor-powered machine are the R G. LeTourneau Company's answer to fast housing construction Both the houses and the method of their construction are highly unconventional The machine which sets the outside steel forms, then lifts and transports the house after the walls, main partition and roof have been monolithically cast, has been named the "Tournalayer" by LeTourneau, but is fast becoming popularly known as the "Bungalow Biddy"

The houses are simple 4-room flat-roof bungalows, 24 × 30 ft in outside dimensions, with 8-ft ceiling height Walls are 5-in concrete, flared out at the bottom to 12 in. as a foundation base The roof is a 7-in slab dished to 5 in at the centre for drainage through a cast-in-place downspout One 5-in longitudinal centre partition is cast with the house Other partitions, dividing the bungalow into two 11 × 11½-ft bedrooms, one 11½ × 16½-ft living room, a 9½ × 12½-ft kitchen and dinette, a bathroom and utility room are cinder block or wallboard on wood studs Concrete floor slabs, containing piping for radiant heating, are poured in place after the house has been moved to its final location

Walls and roof are reinforced with ⅜-in. rods spaced 12 in each way, welded into mats. Openings are then cut in the mats for windows, doors, and two 18-in ventilators in the roof. Additional reinforcing as an eight-sided loop of ⅜-in. bars are placed around the openings. Single ¾-in. bars are set vertically in the corners, along a 4-in parapet roof overhang and in the flared base.

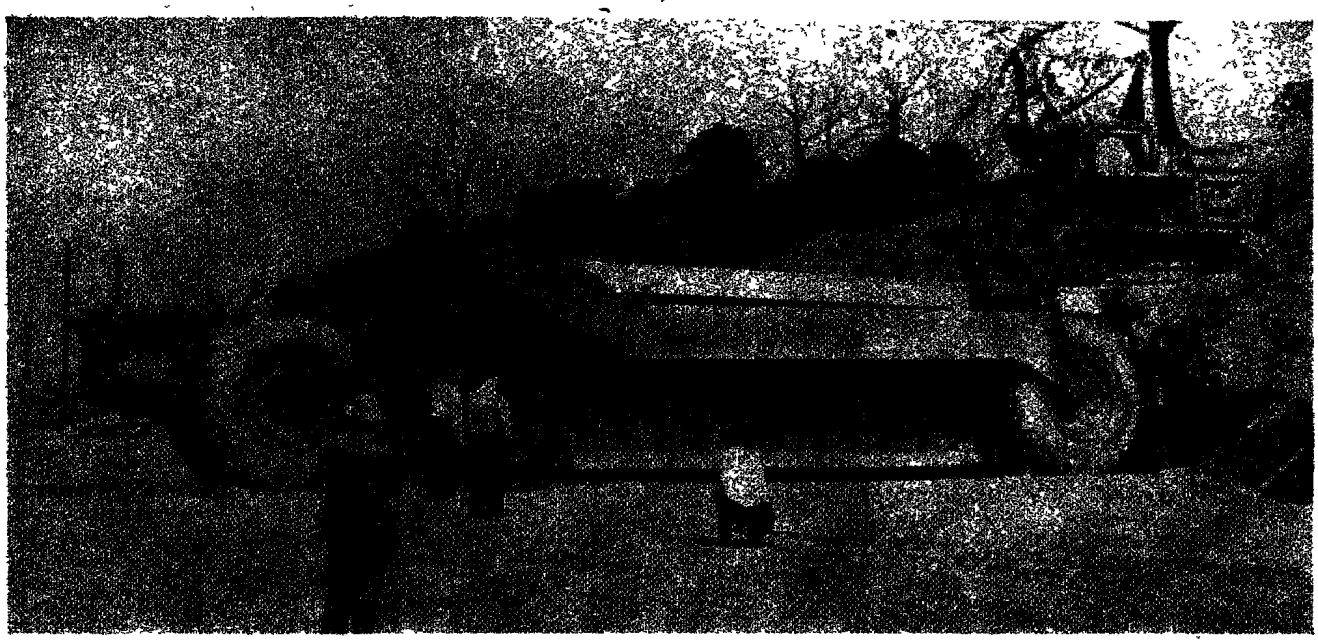
Interior forms, including the roof, are ¼-in steel plate, stiffened with I-beam ribs and quarter-sections of heavy steel pipe at corners and ceiling lines, which leave rounded interior corners in the structure. The forms, joined at midpoint on the sides and at quarter-points at the ends of the house, collapse enough to permit the structure to be lifted clear Access to the inside of the interior forms is through the roof ventilator holes

Outside forms are heavier, of ⅜-in plate, stiffened by I-beam ribs, and mad up in four sections, one for each wal The sections are pinned at the corners b cams that open up for stripping. Th bottom outside flare is reinforced to serv as a lifting beam for the whole structur

Form-setting procedure is to erect th inner forms on a steel casting bed. Th the reinforcing and blocking for th openings are set Next, the outer forr

CABLE HOISTS on Tournalayer (below) hold outside forms high as U-shaped trailer straddl inner form assembly to set outside forms in place. Access to inside to collapse inner for after concrete walls and roof are poured is through ventilator holes in roof.





FORMS ALL SET and ready for concrete. Tournalayer cannot be pulled away until concrete has set for 24 hr. Paver on bank places concrete into forms with boom and bucket.

are dropped over the inside assembly by the Tournalayer The two sets of forms are fastened together by ½-in. threaded bolt ties. After the concrete has been poured, these bolts are driven back out just far enough to clear the inner form shell, but remain in place as shear bolts to lift the structure when the Tournalayer raises and moves the house with outside forms still attached.

The Tournalayer, though huge and ungainly in appearnce, is simply a big two-wheel trailer frame towed and operated by a 200-hp. diesel two-wheel Tournapull tractor Standing almost 14 ft high, it is 51 ft, long and 35 ft wide The U-shaped carrying frame is built up of 28 × 46-in steel box side pieces welded to a 30-in. pipe front piece Tractor and frame are connected through a heavy vertical-pin assembly that bears directly on the tractor's single axle The whole rig is carried on four 30 × 40-in. pneumatic tyres (8 ft 4 in dia.), 34 ply and inflated to 45 psi.

Lifting power is supplied through three electric hoists and 8-part lines, one each mounted on the side frames just ahead of the rear axle, and at the centre of the front frame. Power for the hoists comes from a 50-kva generator on the tractor. The hoists have a lift of 10 ft., ample for the completed house to clear the inner forms, and the outer forms to clear the structure Lower blocks on the hoist system fasten to the flared bottom of the outside forms

Tractor and carrier combined weigh 35 tons. The heaviest lift, that of the house encased in outside forms, is about 120 tons.

Construction Procedure

Setting up the inside forms on the steel casting plate is the first step in manufacturing a house. These forms are painted with a special oil, selected after several experiments to yield the most satisfactory finish of concrete poured directly against the steel plates without liners. Placing of the reinforcing mats, door and window frames, and electric conduits, along with timber shores under the roof, is next Then the machine drops the outer forms over the inner assembly, and the job is ready for pouring

An elevated paver places the concrete directly into the forms with its boom bucket Concrete, vibrated during placing, is made up of lightweight expanded slag (40 lb cu ft) sand and quick-setting cement with a 1 . 2 4 4 mix, a design that gives a minimum 28-day compressive strength of 3,000 psi After 24 hr the inner forms are loosened and the house is ready for transportation

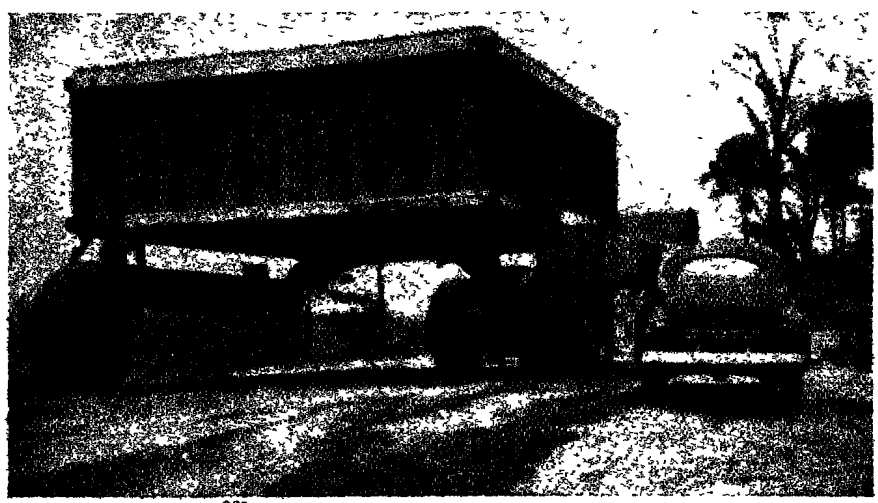
The machine picks up the house, still incased in the outside forms, and sets the structure down at its final site, after which the forms are loosened and lifted clear of the walls Since the flared base of the outside walls acts as a footing, no separate foundations are required. The concrete is cured for three days by water sprays

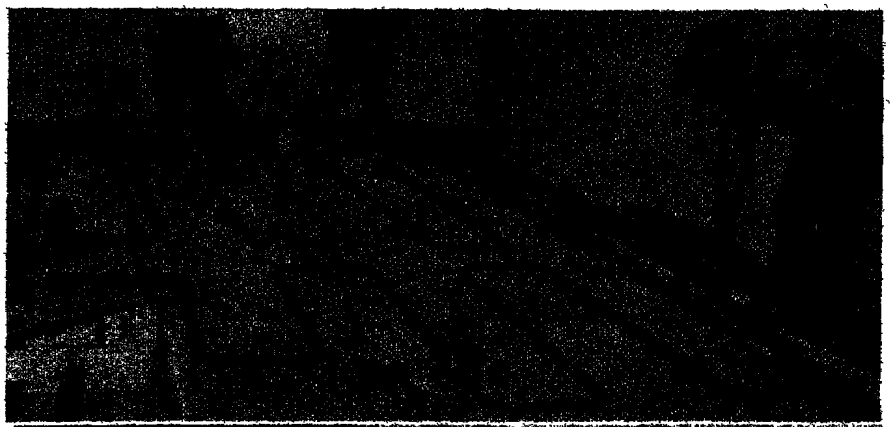
Floors are installed after the house is in place Over a 6-in layer of insulating cinders are placed three thicknesses of waterproof paper, topped by a ½-in layer of lime to counteract possible corrosion of heating pipes by acids in the cinders The radiant heating pipes are set on the lime and are then incased in a concrete floor slab Radiant heating, combined with interior painting with white cement-base waterproof paint, and positive ventilation through slow-speed fans set in the roof ventilators, have eliminated interior wall sweating

Bigger Rigs on the Way

At present LeTourneau is building 100 houses at the rate of three per 5-day

COMPLETED HOUSE SHELL (below) goes for a ride to final location still incased in outside forms Gigantic size of Tournalayer is shown by comparison with automobile.





RADIANT HEATING PIPES are laid in lime bed on paper-covered cinder insulating fill ready for embedding in concrete floor slab. Lime counteracts possible pipe corrosion from acids in cinders. Floor is poured after house is in final location.

week for his employees at the Vicksburg plant with the rig described above Forms for a 24 × 32-ft. house with 9-ft. ceilings are to be made at a new plant at Longview, Texas, and corresponding larger Tournalayers will be manufactured at both Peoria and Vicksburg. LeTourneau does not plan to go into the housing business, but will lease the machine and forms, redesigned for shipping, to contractors

Ray F. Gieszl, general manager of the Vicksburg plant, is directing that plant's housing project, and is responsible for this development of the forms and equipment used there Joe Malik, his assistant, is in direct charge with Raymond Allen supervising the construction operations. —(*With acknowledgments to "Construction Methods"*)



FINISHED PRODUCT is neat, compact four-room bungalow, shown here ready for occupancy. Sash, doors and plumbing are installed after house shell has been placed in final position.

(*Continued from page 224*)



Fig. 4. New construction inside existing abutment arch renews the structure. Left, corbelled arch after sandblasting; centre, reinforcing hung from roof; right, forms in place for dry packing and grouting to make a concrete arch.

to fill the joints between the stones. When drills into the interior, from the downstream face of the south abutment, struck large voids, however, an opening was cut through the perimeter stones, disclosing stone arch construction under the bridge roadway and a deteriorated condition even worse than in the piers.

The interior stone was thoroughly cleaned by sandblasting and what amounts to a new arch was built inside the existing one to reinforce and preserve the original. All of the concrete placed to reinforce the arches was made by forming to the desired shape and then packing with carefully selected crushed stone, followed by filling with mortar grout under pressure.

Concrete arch reinforces masonry

A new wall was built against the arch piers up to the springline, 25 ft. above the base, and in one instance four pilasters were used, surmounted by a concrete beam at the springline. Above the springline and directly against the old stone, a new arch barrel 12 in. thick was placed. Under the corbelled arches upstream a similar shell of concrete was placed to reinforce the structure and serve the additional function of an enclosing wall against which grout could be intruded under pressure to re-establish the strength of the entire masonry wall.

Repair of the Victoria Bridge has been under way for some time and now is nearing completion. Chas. P. Disney, engineer of bridges, Canadian National Railway, Toronto, is in charge of the work, with Mr Redmon, assistant engineer, Toronto, supervising the work in the field.

The methods described in repairing the bridge originated and are patented in the United States and Canada by the Intrusion-Prepakt Co. of Toronto and Cleveland, which is doing the reconstruction work, Wm. Johnson is superintended for the contractors, and H. Henrich is in immediate charge of the field work.—(*With acknowledgements to "Engineering News-Record."*)

DRY PACKING AND GROUTING OF SUBSTRUCTURE SALVAGES 90-YEAR-OLD MONTREAL BRIDGE

CONTENTS IN BRIEF—*Grouting under pressure was successfully used to replace mortar and solidify the masonry of piers built in 1854 for a bridge across the St Lawrence River. Deteriorated stones in the exterior masonry are replaced by packing with crushed stone and intruding grout, with essential admixtures, into it Long approach abutments of stone arch construction have been strengthened by forming a new arch underneath the original by dry packing crushed stone against the arch and forcing grout into it to make concrete.*

COMPLETE retrogression to sand of the lime mortar, in which 111,000 cu. yd. of stone masonry of the Victoria Bridge across the St. Lawrence River at Montreal was laid, has required extensive restoration to salvage the 90-year-old stone piers and arch abutments.

The Victoria Bridge was built in 1854-1859 for the Grand Trunk Railway and for many years was the only fixed crossing of the St. Lawrence River. It has always been one of the most important trade routes in Canada. About 1898 the long superstructure, originally a single track iron "tube," was replaced by pin-connected through trusses, which now carry the double-track main line of the Canadian National Railway. Also accommodated on the bridge, cantilevered outside the trusses, are an important electric railway and a two-lane highway

The load now carried on the substructure is seven times greater than the weight put on it when the bridge was built. Heavier loading, coupled with only point support of many stones due to failure of the mortar, has caused vertical cracks extending the long way of many of the piers This progressive deterioration of the substructure has made it increasingly imperative that the piers either be replaced or repaired

The bridge has 24 spans of about 245 ft. and one channel span of 330 ft. It rests on 24 piers and two abutments, each founded on solid rock and was built in the dry inside floatable caissons or cribs in water 5 to 15 ft deep at low summer flow Since the structure has withstood very swift current and extreme winter conditions, where ice has sometimes piled up over the top of the piers, for about 90 years it must be assumed that the lime mortar and the workmanship originally were of the best quality.

The piers are 22½ × 92 ft at the base The upstream end of each has a pointed cutwater, which slopes 1 to 1 from below low water to above high water to break up the ice The piers are 50 to 60 ft high and at the top are 16 ft wide Originally they were 33 ft long, but were lengthened at the top when the new superstructure was built. Piers at the channel span are 8 ft wider than the others Long enclosed abutments at each of the spans have a sloped face upstream and a vertical wall on the downstream side.

All of the masonry work is coursed ashlar with the top, bottom and ends of each stone of the perimeter courses dressed The exposed faces of the stones are rough dressed and the backs irregular Spaces inside the perimeter courses in the piers are filled with random stones

In addition to loss of bond some of the stones have not withstood the weathering and, particularly in the cutwater ends of the piers, the faces of some large units have disintegrated Failure of the stones has accelerated in recent years, believed due largely to loss of effectiveness of the mortar and subsequent entrance of moisture followed by spalling from frost action.

Stone packing and intrusion grouting

The method adopted for repair of the piers is to pump grout containing essential admixtures into them to make cement mortar of the sand remaining from the original mortar Where large volumes are required, such as where stones require replacement, well graded crushed stone is packed into the area which is then filled with the special grout to make concrete Both of these processes are patented

In repairing the piers, holes are drilled horizontally into them at almost every point of intersection of a vertical joint with the horizontal course beneath it, making a pattern of holes at about 3 × 8-ft centres From just above the water line, holes are drilled into the piers at an angle of 15 deg and 60 deg with the vertical to reach all parts of the base Some of the holes extend into the bed of the river to permit foundation grouting and thus assure full bearing of the base course stones.

Holes to be drilled through several feet of stone are started as large as 3-in dia to allow for reduction in size for successive steel changes to a minimum of 1¼-in dia at the bottom. The exposed joints in the masonry are cleaned out and the interior of the pier washed out so far as practical with high pressure water jets directed into the drilled holes After no more sand can be washed out, the joints are pointed to a depth of at least 8 in with cement mortar to seal all openings except, of course, the drilled holes.





Fig. 1. Grout forced into masonry takes the place of deteriorated mortar in piers of Victoria Bridge at Montreal, built 1854-1859. Right : Typical size of the openings where sand from original mortar has been scraped out of joints in the masonry.

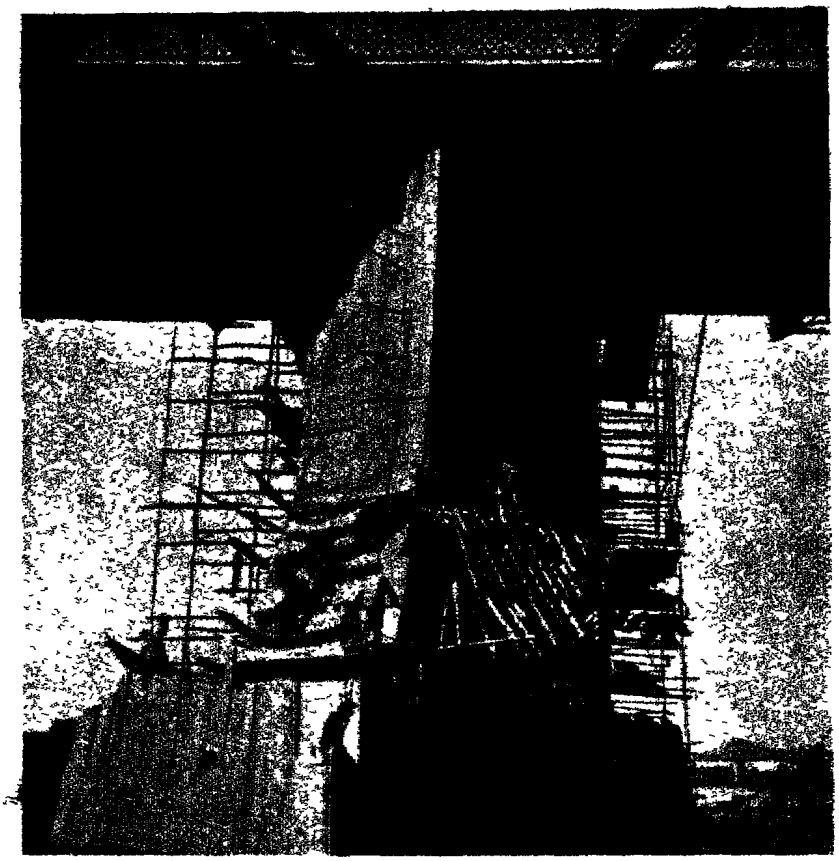


Fig. 2 Broken stones are replaced by cutting back to good material, forming and then dry packing with stone into which grout is forced to make concrete. Note rope net and timber shield to protect masons and divers from the swift current.

Two stage grouting

Grouting is done in two stages, first with cement mortar at low pressure to fill any existing openings, and later with a neat cement grout at high pressure to assure filling any remaining voids in the sand A 1½-in dia pipe, long enough to extend to the full depth, is inserted in successive drilled holes, starting at the bottom, and all of the mortar forced in that the hole will "take" A "packer" around the pipes arranged so that it can be expanded to entirely close the hole, prevents grout from coming out along the pipe Divers examine the underwater portion of the piers and take necessary steps to stop any flow of grout from underwater In the very swift water of the St Lawrence River, a shield around the upstream end of the pier is necessary to protect the divers from being swept down stream

Large volumes filled

For grouting, a mixture of three bags of portland cement and one bag of an admixture intended to reduce shrinkage and delay hardening of the mix are used In the first stage intrusion, 4 to 5 cu. ft. of sand are added to this mixture An air driven, two-cylinder reciprocating pump is used to force the rather fluid mortar into the piers A large amount of grout ordinarily is taken by the piers on the first grouting, one pier having required 9,000 bags of cement The intrusion pipe is placed to discharge at the lowest point of the pier and no attempt is made in this operation to build up pressure other than that required to force the mortar up so that it is extruded from successively higher levels in the pier. Holes above are kept open to vent the air and to show the progress of the filling.

After this grout has had an opportunity to harden, all of the holes are redrilled and a neat grout of three parts of portland cement to one part of the admixture is forced in. Pressures up to 350 psi are used at the foundation rock level, but this is reduced to relatively small amounts near the top to prevent cracking the capstones of the piers. Comparatively small quantities of cement are used in the regrouting

Displaced stones are replaced and broken stones repaired by filling the areas with crushed stone, well graded to secure a minimum of voids, then forcing in grout under pressure to coat the stones and form the mass into concrete Broken stones are chipped out to a depth of a few inches, a wood form is built against the pier to the shape of the stone to be replaced and the space inside packed tightly with stone Forms are held in place by expansion anchors and are caulked all around so that a pressure head can be applied to force in the grout. Caulking is accomplished with soft rope, tacked to the form in such a position that it leaves a mark similar to the pointing between the stones.

Repairing the abutments

The abutments of the bridge presented an entirely different problem from that of the piers Details of their design were not known to the repair crews, but they appeared to be solid masonry with an interior rubble fill that was settling, presumably due to deterioration of the lime mortar. Prescribed treatment was to intrude grout

(*Continued on page 222*)

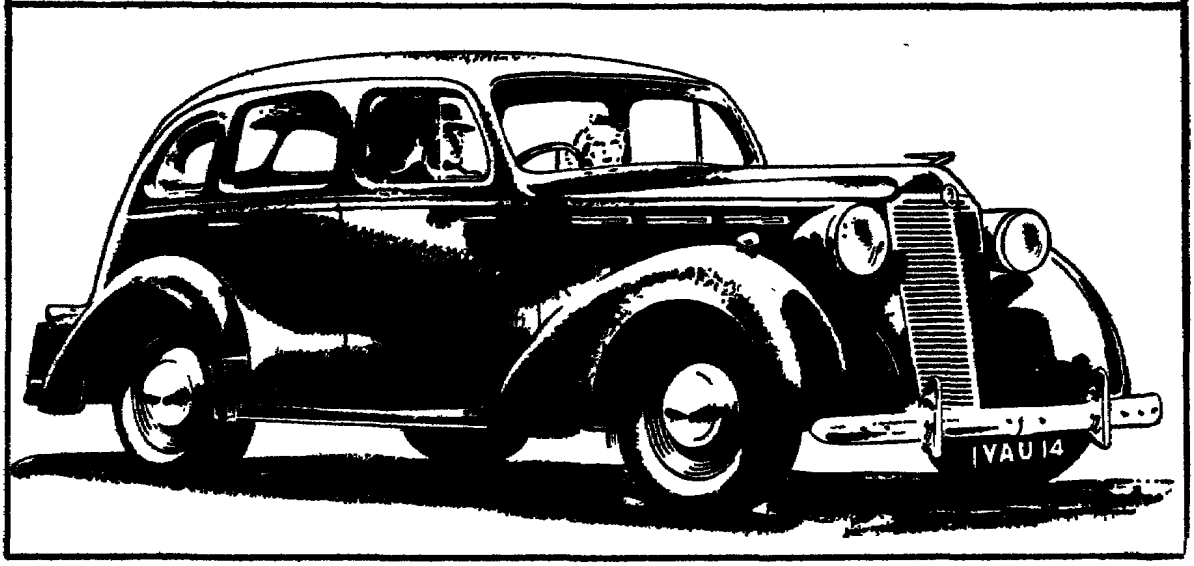


Fig. 3. After weathering, replacement "stones" blend unobtrusively into the structure.

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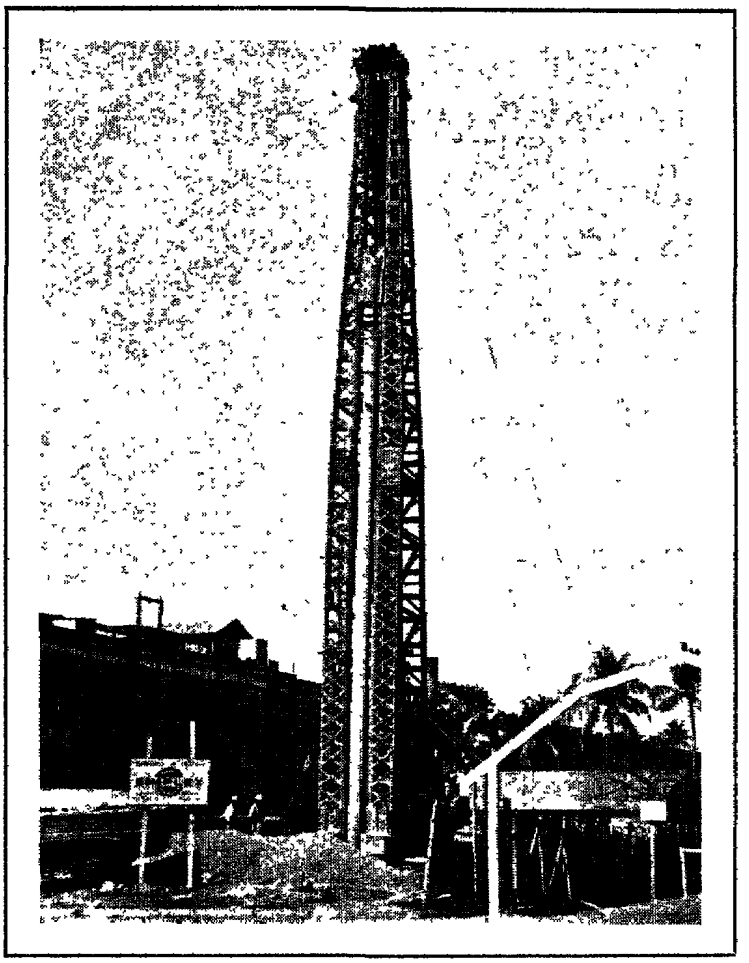
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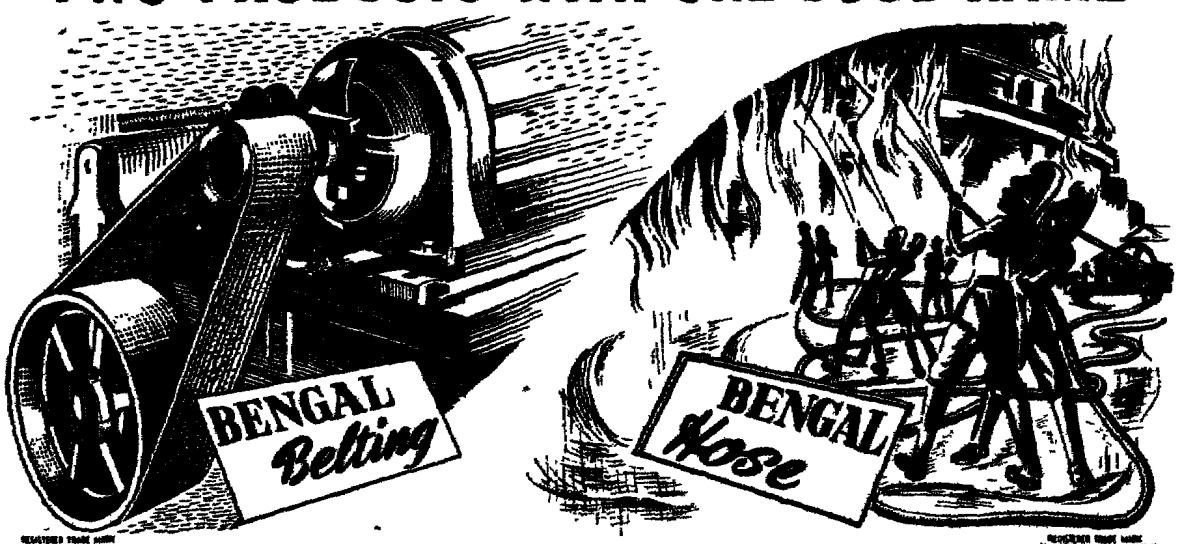
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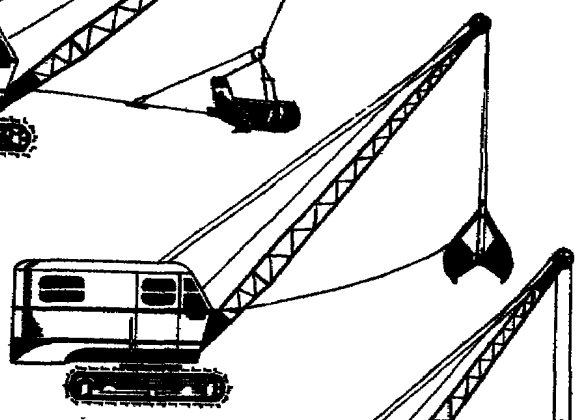
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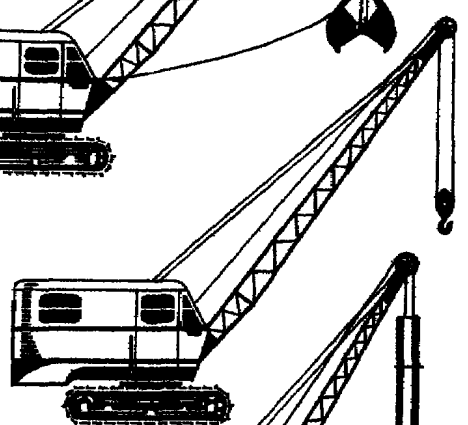


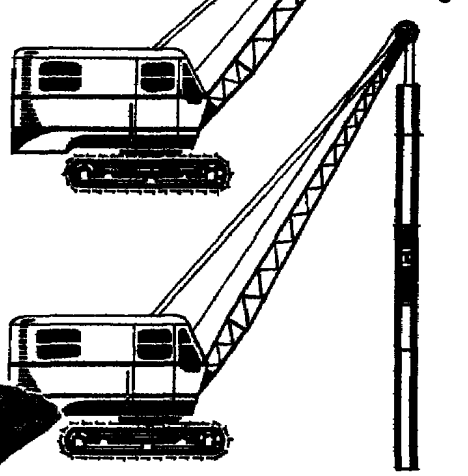
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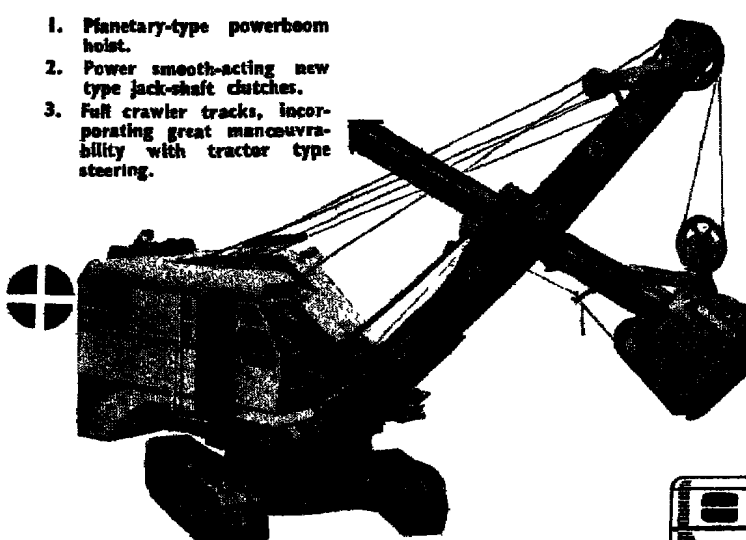


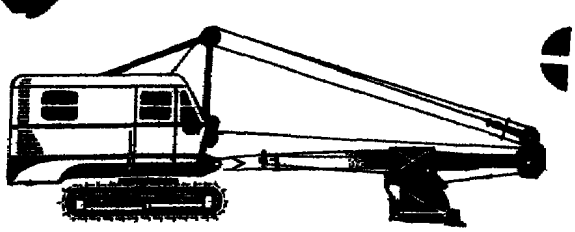
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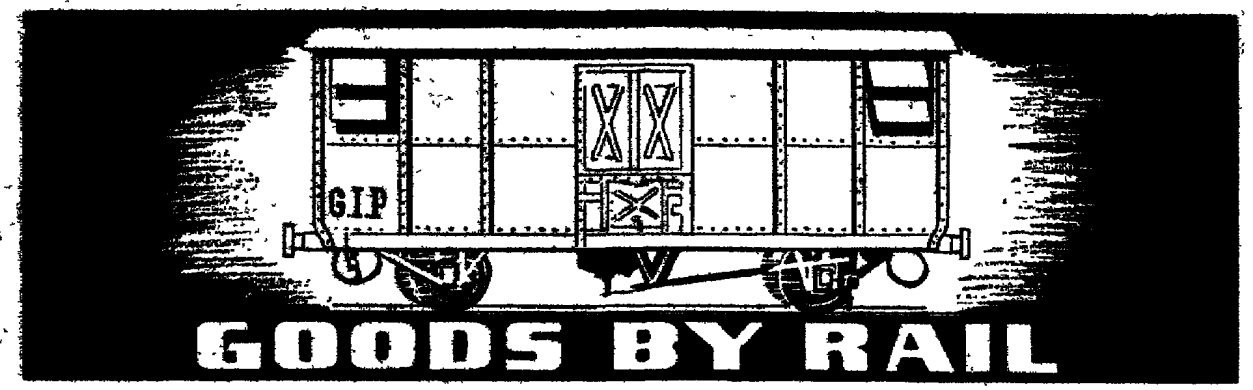
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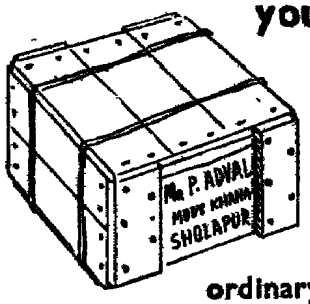
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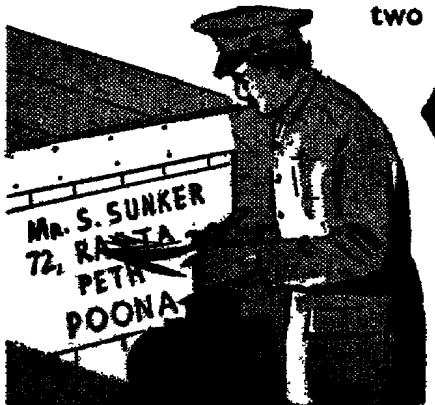
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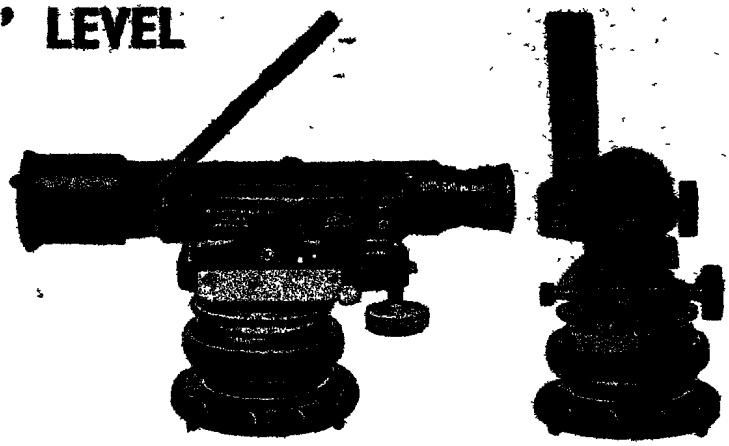
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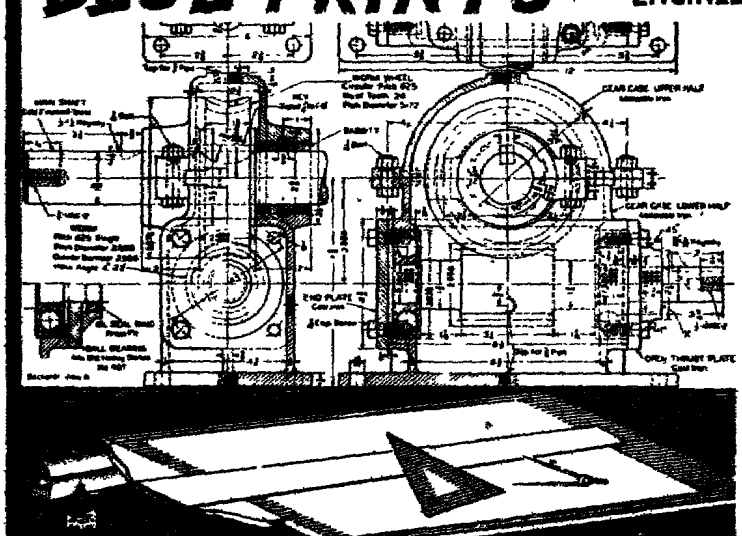
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JARANWALA
JHANG MAGHIANA
JHELUM
JOGINDERNAGAR
JULLUNDER CITY & CANTT.
JUTOGH
JWALAMUKHI ROAD
KHEWRA
KABUL
R. S. Chaman & Peshawar.
KABUL RIVER
KAMALIA
KALAKH
KALKA
KALABAGH
KALLUR KOT
KAMOKE
KANGANPUR
KANGRA
KAPURTHALA
KARTARPUR
KAROR
KASAULI
R.S. Kalka.
KHAIRABAD
KHUSHAB
KASUR
KHANEWAL
KHAROR PAKKA
KOT RADHAKISHEN
KANDAGHAT
KUNDIAN
KHANNA
KHANPUR
KHASA
KOT ADU
KHARIAN
KILA RAIPUR
KOHAT
KOT KAPURA
KOT LAKHPAT
KURALI
LAHORE & LAHORE CANTT.
LAKHEWALI
LAKMAREWAT
LALAMUSA
LALIAN
LODHRAN
LOHIAN KHAS
LEIAH
LUDHIANA
LYALLPUR
MAGLMOD GUNJ ROAD
MAGHARWAL
MALSI
MAJITHa
MALAKWAL
MALERKOTLA
MANHU
MALOUT
MALSIAN SHAKOT
MANDI BAHAUDDIN
MANDRA
MANSA
MARH BALOCHAN
MANDI BUREWALA
MANDI STATE
MANGWAL
MARDAN
MANU KANJAN
MAUR
MAINWALI
MIRPUR
R. S Surat Alampir
MIRAN SAHIB
MOGA
MOGHALPURA
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MULLANPUR
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MUZZAFFARGARH
MURIDKE
MIANCHANNU
MINCHINABAD
MIRAN SAHIB
NAKODAR
NANKANA SAHIB
NARANG
NAROWAL
NAWANSHAHR DOABA
NOWSHERA
NURKOT
NURMAHAL
NURPORE ROAD
NAGROTA
OKARA
PALAMPUR
PARACHINAR (*Via Thal*)
PASRUR
PATHANKOT
PAKPATTAN
PATTI
PINDIGHEB
PINJOUR
PATTOKI
PESHAWAR
PHAGWARA
PHILLAUR
PHULERWAN
PIRMAHAL
PIND DADAN KHA
PIPLAN
QAIDAN
QILLA SHEIKHUPURA
QILA SOBHA SINGH
RAHON
RANBIRSINGHPURA
RAIKOT (*R. S. Mullanpur*)
RAHWALI
RAIWIND
RAHIMYARKHAN
RAJA JANG
RANBIR SINGHPURA
RORANWALA
RAWALPINDI
RENALA KHURD
RUPAR
RURALA ROAD
SADIKABAD
SAMBRIAL
SAILA KHURD
SANAHWAL
SANGLA HILL
SILLANWALI
SARA ALAMGIR
SARGODHA
SARNA
SATNAUR BADESRON
SHAH ALAM
SHAHDARA
SHARAKPUR
(*R. S. Shahdara*)
SHAHPUR SADAR
SHAH CHAURASI
SHORKOT ROAD
SHUJABAD
SIALKOT
SIHALA
SIMLA
SIRHIND
SOLON
SUKHO
SULTANPUR LODHI
SUKHEKE
SRINAGAR
R. S Handian or Rawalpindi.
TALAGANG (*R. S. Chakwal*)
TALWANDI
TARN TARAN
TANDA URMAR
TANDIANWALA
TANK
TAXILA
THERI SANSI
TEHSIL SHAKARGARH
THABALKE
THAL
TOBA TEK SINGH
VEHARI
VERKA
WAZIRABAD
WARBURTON
WASAWEWALA

NEW DELHI AREA :—

ACHNERA
AGRA
ALAL
ALIGARH
AMROHA
AMBALA CITY
AMBALA CANTT.
ATRAULI ROAD
BABRALA
BARHAN
BARARA
BARNALA
BAGHPAT ROAD
BAH (*R. S. Balaugani*)
BAHADURGARH
BAHJOI
BARETA
BASIKIRATPUR
BASIKRATPUR
BEGAMABAD
BHIWANI
BHUCHCHU
BALLABGARH
BARAUT
BUDHLADA
BULANDSHAHR
BIJNOR
BRINDABAN
CHANDAUSI
CHANDPUR SIAU
CHHINTANWALA
CHOLA
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DALMIA-DADRI
DIBAI
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DELHI
DELHI SHAHDRA
DEOBAND
DANKAUR
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GANAUR
GANGOH
GARHMUKTESAR
GHAZIABAD
GHABAUNDA
GOT
GOHANA
GUBGAON
GULAOTHI
HALDAUR
HANSI
HARDWAR
HAPUR
HATHRAS
HISSAR
HASANPUR (*R. S. Gajraula*)
JAGADHRI
JAKHAL
JALESAR ROAD
JAWALAPUR
JIND CITY
JULANA
KALANWALI
KANTH
KANDHLA
KAURARA
KHATAULI
KHEKRA
KHURJA
KARWAL
KAITHAL
KASHIPUR
KASIMPUR KHERI
KOSI KALAN
KOSLI
KOTDWARA
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SANGRUR
SASNI
SUNAM
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SEOHARA
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SHAHABAD MARKANDA
SHAMLI
SIRSA
SONEPAT
SIMBHAOLI
TAPA
TAPRI
TARWARI
THANA BHAWAN
THANESAR
TUNDLA
TOHANA
UCHANA
UKLANA

CAWNPORE AREA :—

ACHALGANJ
ACHHALDA
ACHAULI
AHIMANPUR
AHRAURA ROAD
AJGAIN
AJODHYA
AKBARPUR
ALIGANJ
ALLAHABAD
AMETHI
AMOUSI
ANJHI
ANUPGANJ
AONLA
ASAFPUR
ATARRA
AURAIYA
AUNRIHAR
AZAMGARH
BABHNAN
BACHRAWAN
BADLAPUR
BADSHAHPUR
BANERI
BAHRAICH
BAHRAMGHAT
BAISWARA
BAITALPUR
BAKSHA
BALAMAU
BALLIA
BALRAMPUR
BAMIANA
BANBASSA
BANDA
BANGAR MAU
BARA BANKI
BAREILLY
BARHAJ BAZAR
BARHNI
BARHALGANJ
BASAI
BASHARATGANJ
BASTI
BELLARRIN
BELTHARA ROAD
BENARES CANTT.
BENARES CITY
BEWAR
BHADOHI
BHARTHANA
BHATPARRAN
BHATNI
BHITAURA
BHITI
BHONGAON
BIJAURA
BILGRAM
BILPUR
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BISALPUR
BISWAN
BODARWAR
BRIDGMANGANJ
BUDAUN
BURHWAL
CAMPIERGANJ
CAPTAINGANJ
CAWNPORE
CHANDAULI MANJHWAR
CHANDWAK
CHAURICHURA
CHHITAUNI
CHHIBRAMAU
CHILBILA
CHILH
CHILWARIA
CHIRGAON
CHIT-BARAGAON
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DOHRIGHAT
DULAHPUR
ETAH
ETAWAH
FARRUKHABAD
FATEHGARH
FATEHPUR
FYZABAD
GAHMAR
GANJDUNDWARA
GAURIGANJ
GHAZIPUR
GHOSI
GHUGHLI
GOLA GOKARAN NATH
GONDA
GORAKHPUR
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GURSHAHAIGANJ
HAIDARGARH
HALDWANI
HANDIA KHAS
HARDOI
HARGAON
HARNAMGANJ
HARPALGANJ
HARPALPUR

Cawnpore Area (Contd.)

JAHANAGANJ ROAD
JAIS
JAKHANIA
JALALPUR
JANGHAI JN.
JANGIGANJ
JARWAL ROAD
JASWANTNAGAR
JAUNPUR
JEONATHPUR
JHANSI
JHINJHAK
JHUSI
KABRAI
KACHHLA GHAT
KACHHWA ROAD
KADIPUR
KAIMGANJ
KALPI
KAMALGANJ
KAMALPUR
KANAUJ
KANCHAUSI
KARARI
KARENGI
KARWI
KASGANJ
KASHI
KATHGODAM
KATHKUIYAN
KATKA
KAWAPUR
KEORIPUR
KERAKAT
KHADA
KHAGA
KHALILABAD
KHATEMA
KHETA SARAI
KHORASAN ROAD
KICHHA
KIDIHDAPUR
KOHNDAUR
KONRH ROAD
KOPAGANJ
KUNCH
KUNRAGHAT
KUSMHI
LACHHMIPUR
LAKHIMPUR KHERI
LAKSHMIGANJ
LALGANJ
LALITPUR
LAR ROAD
LUCKNOW
MADHOGANJ
MADHOSINGH
MAHMUDABAD
MAHOBA
MAHOLI
MAHPUR
MAINPURI
MAKRANDPUR
MALIHABAD
MALIPUR
MALLAWAN
MANAURI
MANDAH ROAD
MANKAPUR
MANPUR NAGARIA
MARHRA
MARIAHU
MASKANWA
MAU AIMA
MAU NATH BHANJAN
MAURANIPUR
MEJA ROAD
MIRANPUR KATRA
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MISRIKH TIRATH
MOGHAL SARAI
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MUHAMMADABAD GOHNA
MUNDERWA
MUBAFIRKHANA
NAGARIA SADAT
NAIPALGUNJ ROAD
NANDGUNJ
NANPARA
NAUTANWA
NAWABGUNJ GONDA
NIBKARORE
NOWGARH
ORAI
ORCHHA
PACHPERWA
PADRAUNA
PARIAWAN
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PHAPHUND
PHARENDA
PHARIHA
PHEPHNA
PHULPUR
PIPRAICH
PITAMBERPUR
PILIBHIT
PUKHRAYAN
PUNCH
PURANPUR
RAGAUL
RAGHURAJ SINGH
RAI BARELI
RAJA-KA-RAMPUR
RAJA TALAB
RAJAWARI
RAMKOLA
RANI-KI-SARAI
RASAHLI
RASRA
RASULABAD
ROTI
RICHHA ROAD
RISIA
RUDAIN
RUDAULI
RURA
SAADAT
SAFDAR GANJ
SAFIPUR
SAHATWAR
SAHAWAR TOWN
SAHJANWA
SAIDPUR BHITR
SAIYEDRAJA
SAKALDIHA
SALIMPUR
SANDILA
SARAIMIR
SARDARNAGAR
SARJU
SHAHGANJ
SHAHGARH
SHAHJAHANPUR
SHAMSHABAD
SHEIKHUPUR
SHIVPUR
SHOHRATGARH
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SIDHAULI
SIKANDARPUR
SIKANDRA RAO
SISOTARGHAT
SISWA BAZAR
SITAPUR
SORON
SULTANPUR
SURAIMANPUR
SURIAWAN
TAHSIL DEORIA
TAHSIL FATEHPUR
TAKIA
TAMKOHI ROAD
TANAKPUR
TANDA
TARAON
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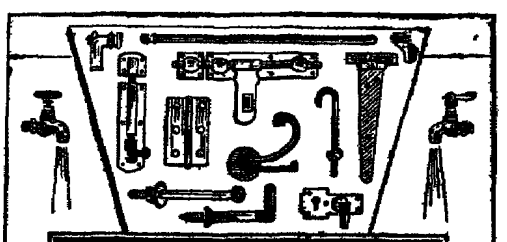
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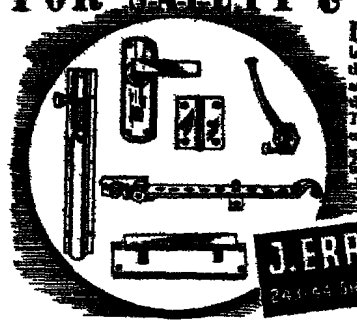
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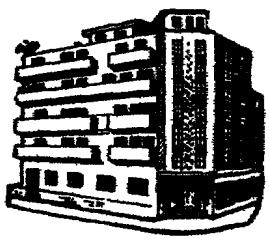
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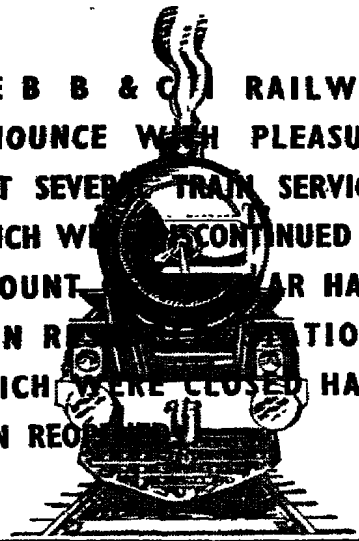
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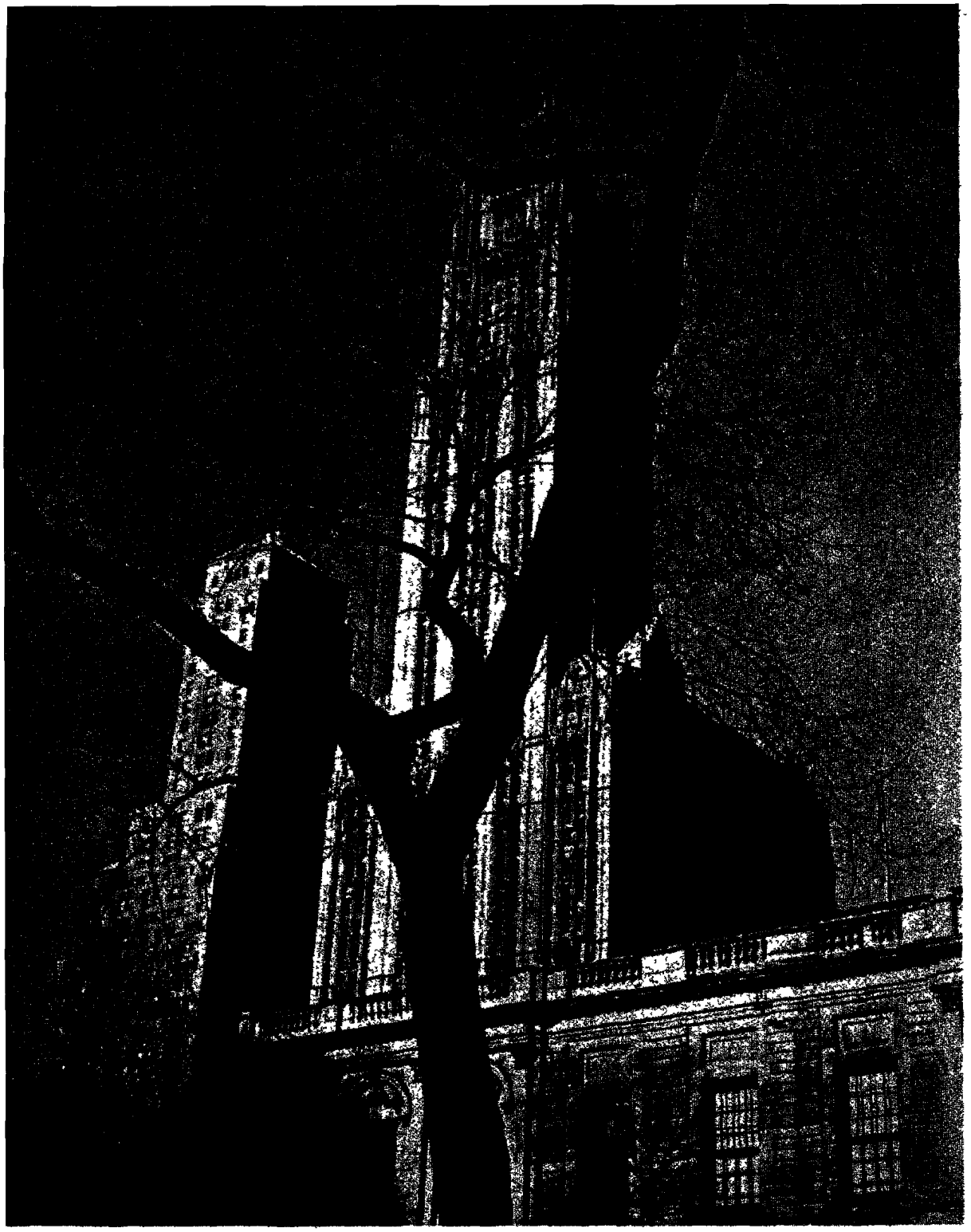
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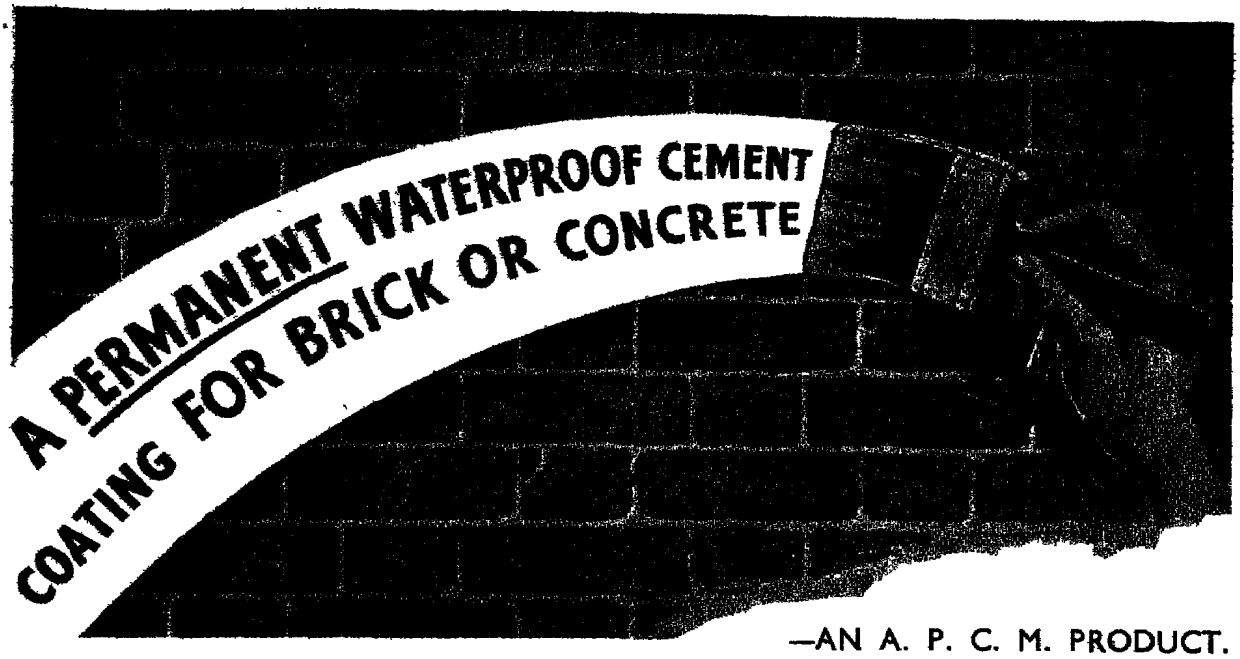
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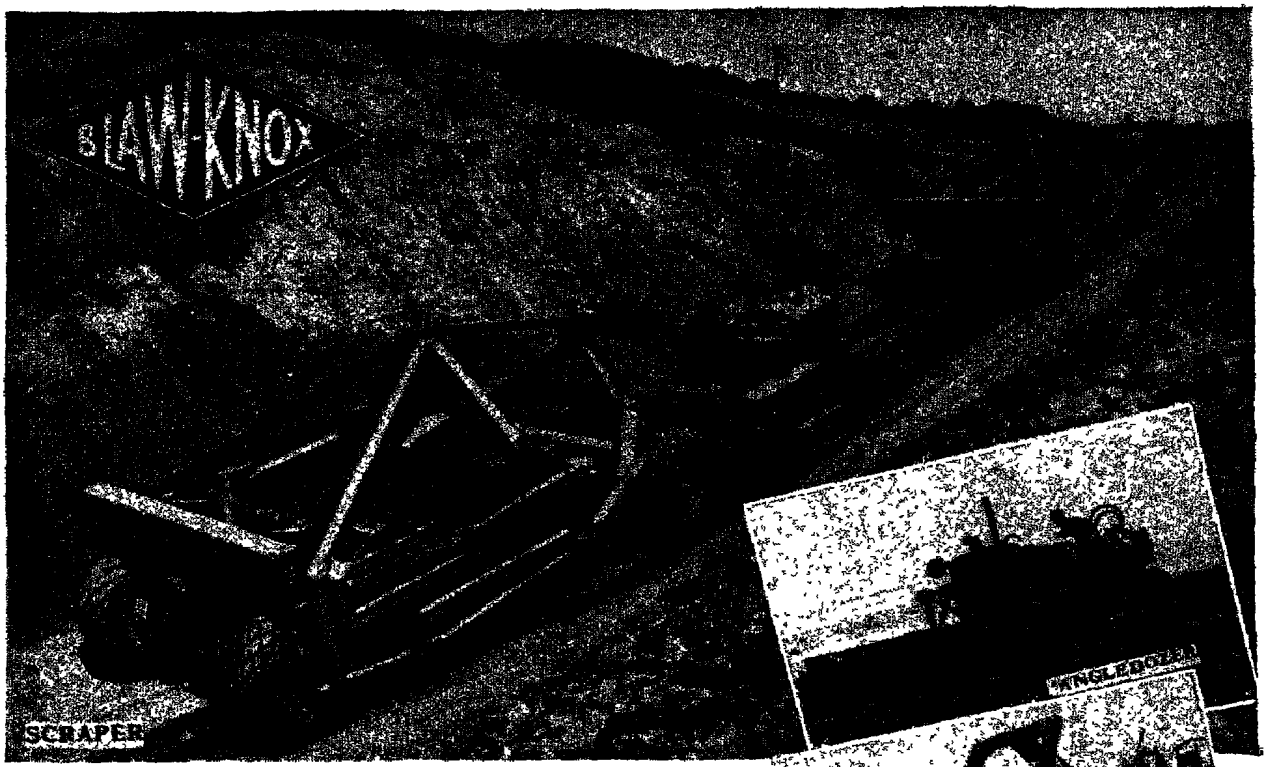
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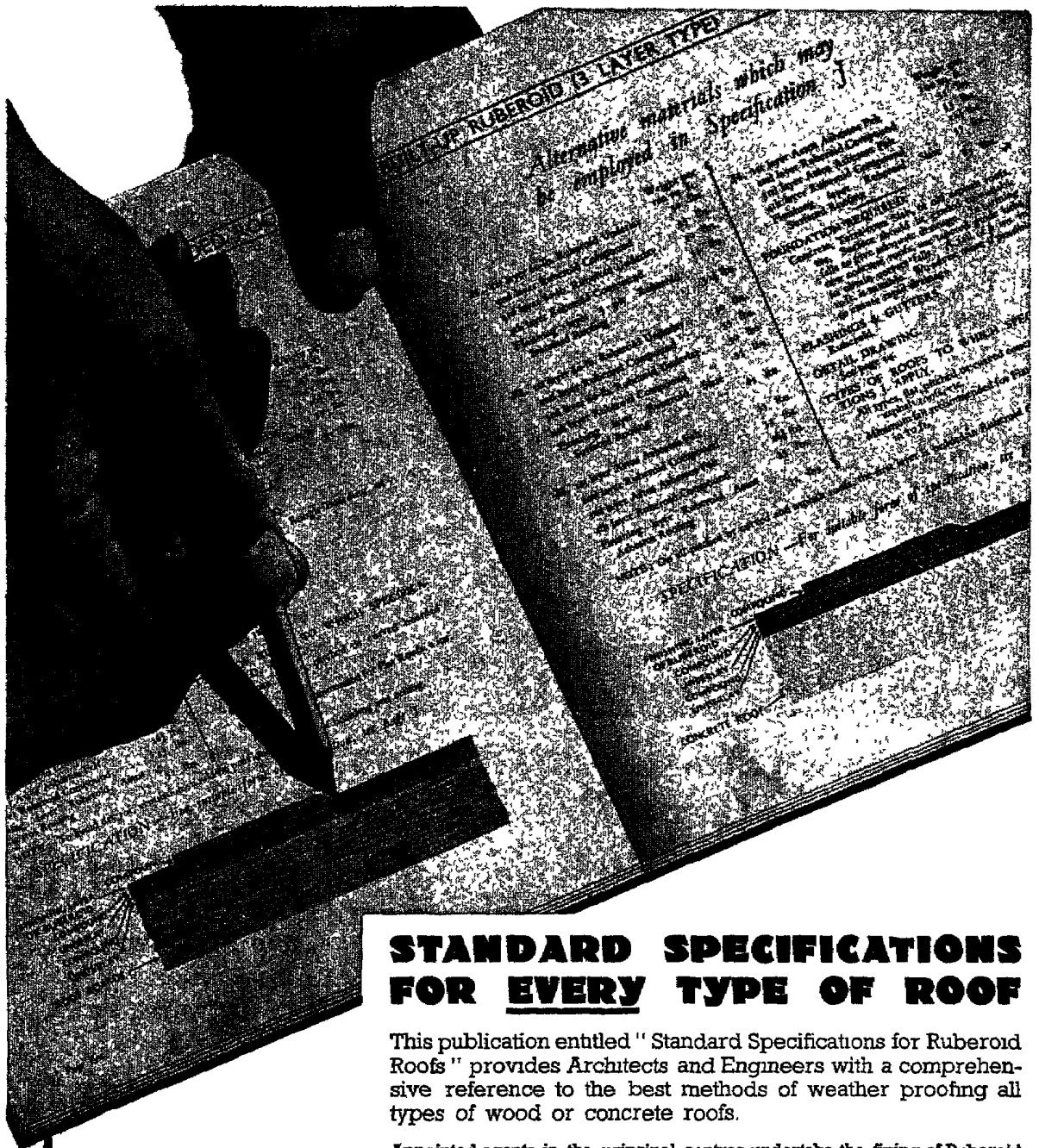
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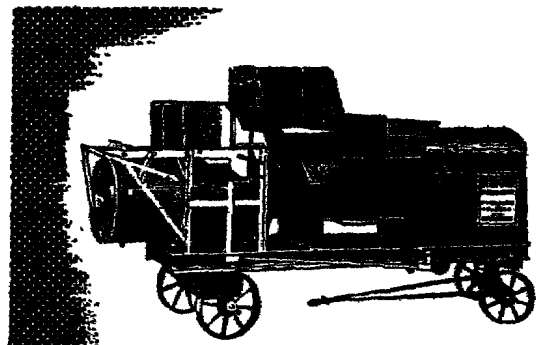
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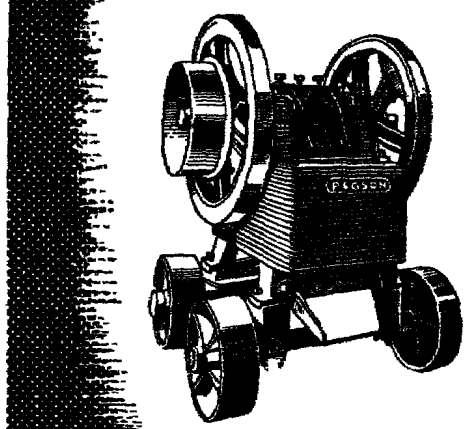
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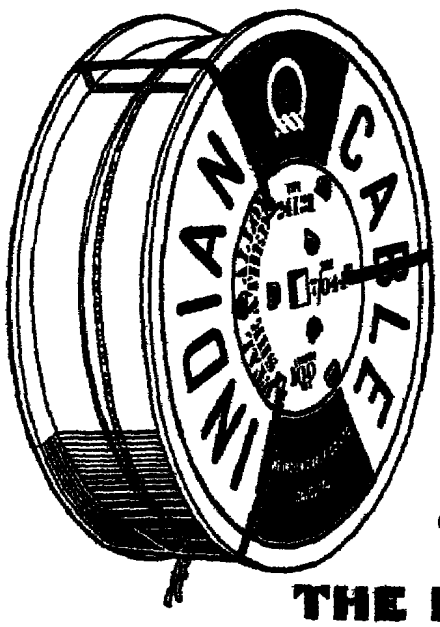
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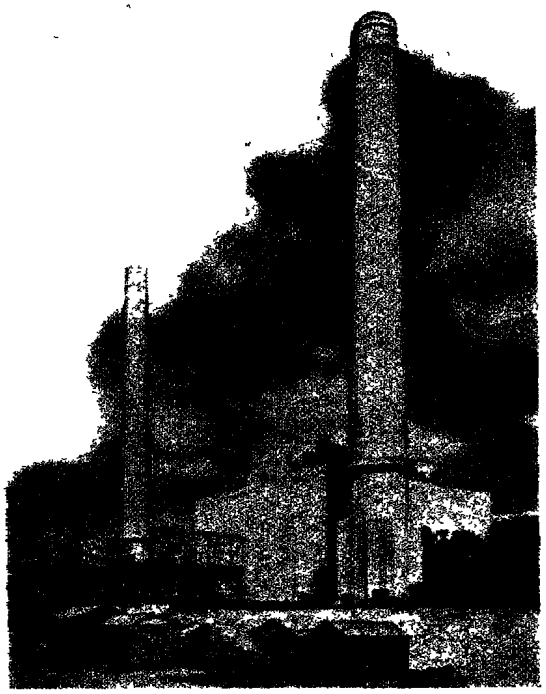
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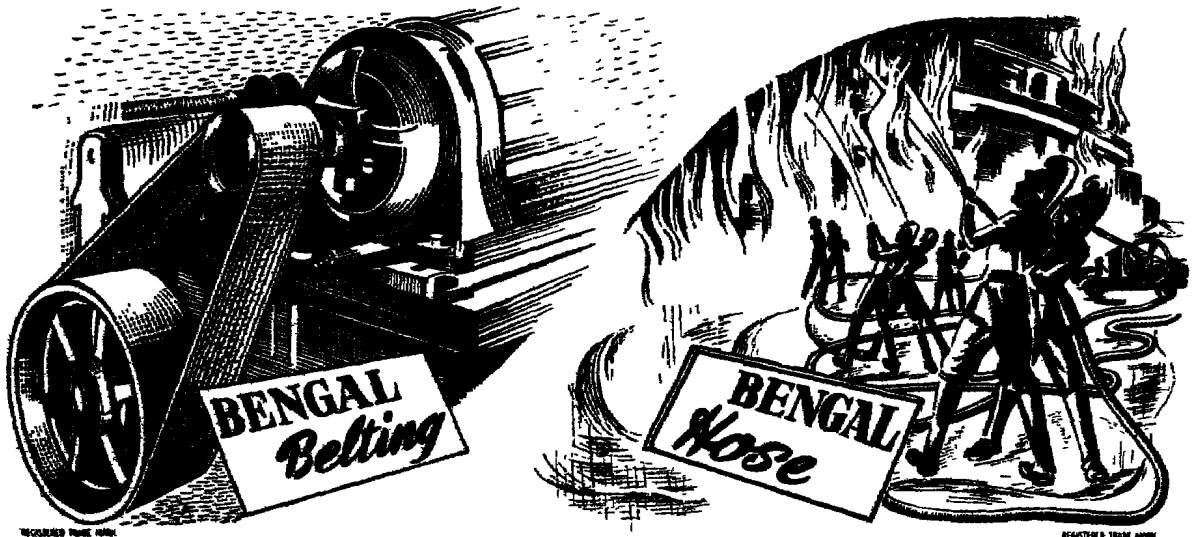
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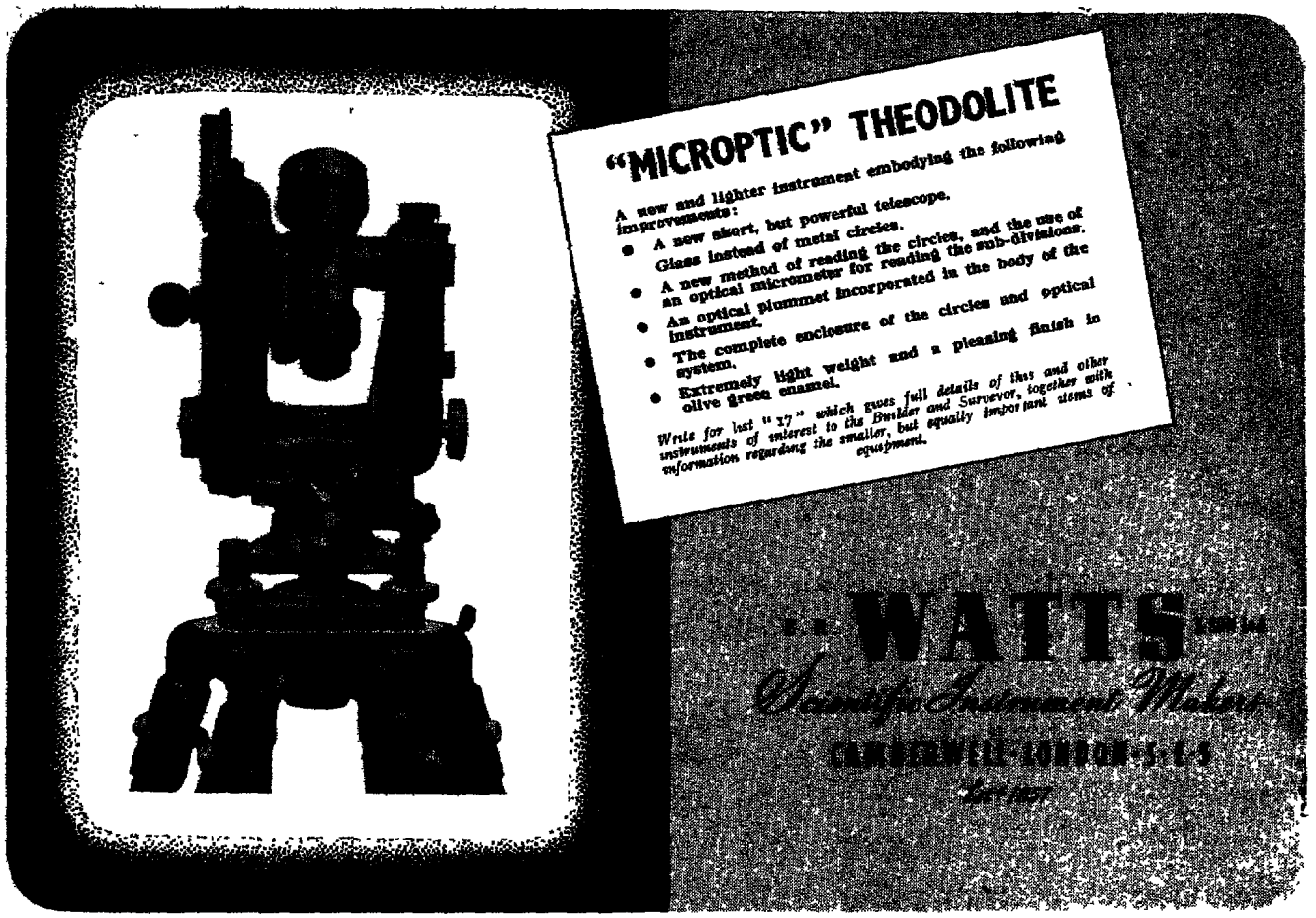
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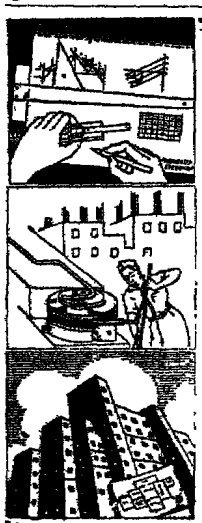
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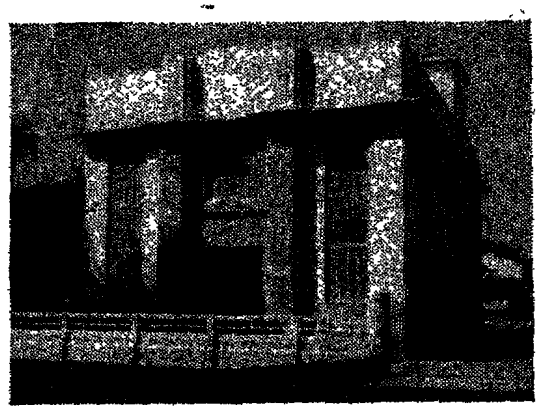
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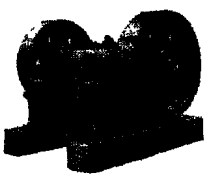
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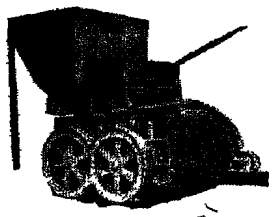
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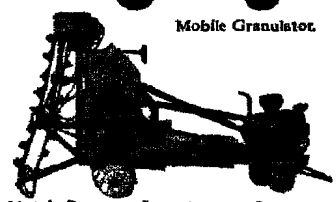
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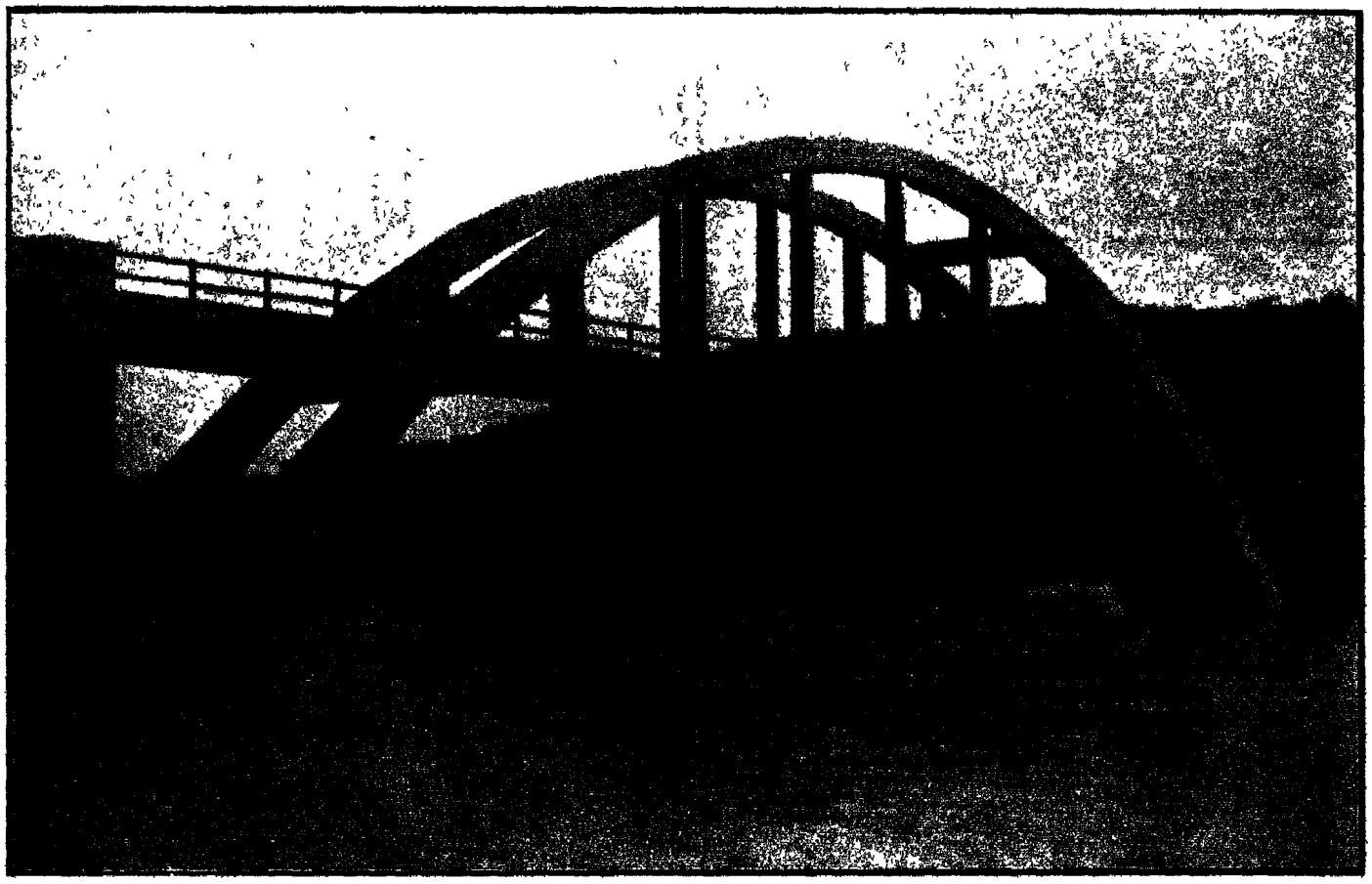
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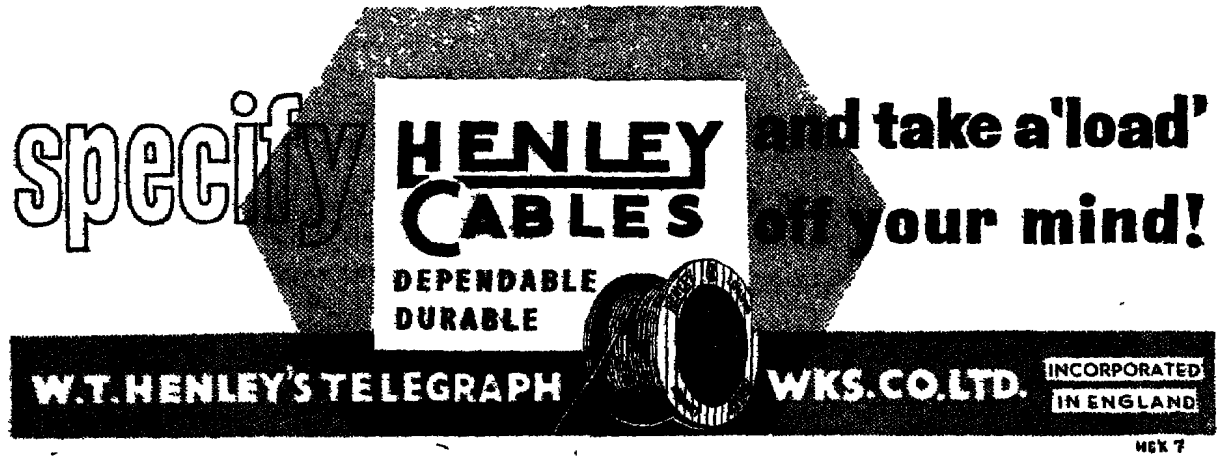
H. S. Batliwala, B.E., M.I.E. (India), M. I. Struct. E., M.I.H.E. (London).

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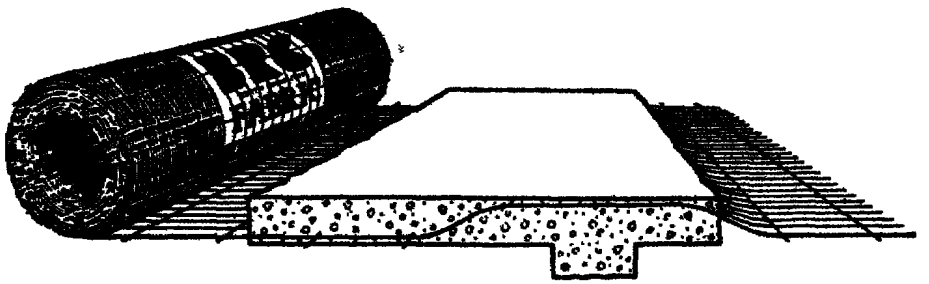
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The formwork which supports the wet concrete until it has set is made up from timber, steel sheets and angles, or cement-asbestos sheets. When the beams are of reinforced concrete the bearers for the floor forms are usually supported on the beam-side forms. When encased steel beams are used the construction may be similar, but it is customary to suspend the formwork from the beams when these are of steel sections.

It is important that the concrete be mixed without excess water coming up to the screeded surface of the work, so that the full strength and hardness of the concrete may be obtained.

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EDITORIAL NEWS & NOTES



CONTROLLED CONCRETE PAVING CONSTRUCTION

WE have pleasure in publishing below an extract from the *Contract Journal*, London, dealing with Controlled Concrete Paving Construction. It will be seen from the article that the saving on materials between a co-efficient of variation in concrete strength of 10 per cent. and one of 18 per cent. is 5s a cubic yard. It has been known to concrete engineers for many years that such a saving is possible. Now in the Heathrow Aerodrome in London Controlled Concrete has become an accomplished fact and we very much hope that all engineers in this country, and there must be many of them working on Aerodromes and Air Ports, will give earnest attention to this matter, not only as a matter of professional pride but for the economic saving which will be entailed.

It has long been known that the regulation of water in the mixing of concrete tends to better concrete, but until vibration became popular it was difficult to reconcile good concrete with a very harsh mix. With mechanical vibrators this trouble has, to a large extent, been overcome and we feel we cannot stress too strongly the value of vibration for all concrete work whether plain or reinforced and we once again remind our readers that every particle of water over and above that required by the cement itself must eventually leave the concrete and where water which is an incompressible substance leaves concrete it necessarily leaves hollows or voids behind which cause cracking and the attendant evils which are so well known.

Generally speaking controlled concrete means good concrete and as we see from the article below it also means cheaper concrete into the bargain.

While it is admitted that where vast quantities of labour are available it should be cheap it has been realised by engineers and contractors more and more that labour in this country is no longer cheap and while we do not expect a violent change over to mechanical methods we shall have to aim at cutting out certain controls which are at present left to human agency and substituting mechanical controls which are unfortunately much more reliable. To whatever extent machinery is introduced in this country there will always be work for the people provided wages and costs are not boosted up to uneconomic heights by overanxiety on the part of labour. There is such a thing as killing the goose which lays the golden eggs and if labour throughout the country insists on overplaying its hand it must necessarily suffer the setback consequent on increased mechanization or reduced activity in industry and public works.

"Although the London airport—the more prosaic and ubiquitous name that has superseded the older one of "Heathrow"—has witnessed many changes during the past months, perhaps none is more significant than a minor revolution of method in construction, or, perhaps better, two revolutions.

First, there is the implementation of a long-advocated policy of controlled manufacture of concrete plus controlled construction of paving, and second, there is the setting-up of an intricate and smooth organisation which necessarily included top-rate maintenance which probably equals, if not excels, anything of its kind in the world.

The results of this large experiment in controlling every stage of concrete production should, by the excellence of the work itself, convince those that remained sceptical of the value of the extra care and expense incurred. For not only has a high-grade concrete been achieved, but, what is most important also to the contractor and the responsible engineers and authorities, the financial saving on the contract cost, apart from the probable avoidance of paving failures in the future and excessive maintenance, may well exceed the cost of the implementation of the control. At the meeting of the Institution of Civil Engineers when the paper on the Heathrow concrete work was submitted by Messrs. G. Graham and F. R. Martin, Dr. Glanville, of the Road Research Laboratory, put the saving on materials between a co-efficient of variation in concrete strength of 10 per cent. and one of 18 per cent. as 5s. a cubic yard, assuming cement to be £3 a ton and aggregates at 10s.

So the final result seems to be better concrete at a smaller cost. The aim of control is to reduce the co-efficient of variation, and the figure of 9.5 per cent. achieved at Heathrow is considered excellent and to approach laboratory precision. The compressive strength demanded for these runways was 4,000 lb. a square inch at 28 days, and the average compressive strength obtained was over 5,200. With a co-efficient of, say, 18 per cent., that is, with much less control, a much higher average strength must be used for which considerably more cement is needed.

Here, then, under the Air Ministry Directorate of Works is the field corroboration of research and the theory of laboratory control, and the results obtained may well become a temporary guide-book for those who will have control of the large road, airport and other civil engineering schemes of the next few years, even though local conditions may not be so favourable to meet the requirements of the specifications. The results also suggest that further investigation might profitably be made into the extension of large-scale weigh-batching plus other controls for a number of smaller works in the same area on the ready-mix and truck-mix methods."

DESIGN OF FOUNDATIONS FROM STAND-POINT OF SOIL MECHANICS

By N. S. RAMASWAMI

Notations used

c	unit cohesion
d	diameter
E	modulus of elasticity
H	height
Ka	active Rankine value
Kp	passive ,, ,,
L	Length
P	Concentrated load
p	unit load, pressure per unit area, stress
q	Ultimate bearing value of the soil
r	radius
z	depth under the ground surface
α	general designation of an angle
γ	unit weight of the soil
Δ, s	Settlement
μ	Poisson's ratio
σ_z	Vertical pressure
ϕ	Angle of internal friction

1. Introduction

It would hardly be an exaggeration to say that the design of a foundation for any structure is productive of the greatest doubt in the mind of the designer and yet in many cases, the subject receives the least detailed considerations.

In India Soil Mechanics has attracted so little attention among practising Engineers as opposed to research workers, that it may be worthwhile to give a short account of the theory and experimental work Professor Terzaghi has pointed out that the permissible pressures on the ground given in Text books and building codes omit any mention of the areas and depths of foundations to which they are applied and that in this respect the figures can be compared with tables giving the breaking loads of beams without mentioning the lengths of the spans. This is a legitimate comparison but one, which is not very helpful when one remembers the contradictory notes published from time to time. Following tradition a designer may decide to make loading tests on the site, but the results so obtained may not necessarily be applicable to the type of structure to be erected.

Modern scientific methods are producing great accuracy in design and execution, of the structures and the form of structures is changing rapidly with the improvement of scientific knowledge and it is noticeable that the massive masonry building with the heavy footings is being abandoned in favour of the framed structure with isolated supports. The change behoves the designer to make a study of the design of foundations.

Traditional design of foundations fail to consider the following factors :—

1. The type of structure.
2. The shape and size of footings.
3. The nature of the Strata upon which the structure is to be founded and
4. the effects of time on soil.

The design of foundations is mainly a problem, which has to be decided for each individual site. It has often been found in experience that structures which had been designed with a constant itensity of pressure on the footings have been subject to uneven settlement. The effects of uneven settlement in a modern-framed structure is productive of irreparable damage. It is of primary importance, that the behaviour of the soil upon which the structure is to be founded, should be examined under conditions comparable with those which will apply in the finished structure Foundation shall therefore be designed in such a manner that the settlement which is inevitable shall not exceed permissible limits and shall have no deleterious effects upon the structure.

2. Stresses under a foundation

If a concentrated load P acts on an infinite elastic body as shown in figure 1, the stress (p) at any point say X (figure)

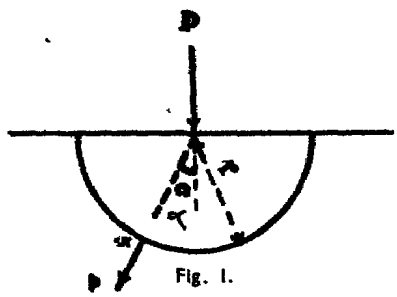


Fig. 1.

is given by the following equation due to Boussinesq

$$p = \frac{3P}{2\pi r^2} \cos^2 \alpha \quad \ldots\ldots\ldots\ldots\ldots (1)$$

Although soil is not an elastic homogeneous body, numerous experiments have prescribed that formula of the type

$$p = \frac{(n-2)P}{2\pi r^2} \cos^{n-3} \alpha \quad \ldots\ldots\ldots\ldots (2)$$

is applicable to all soils and that for clay a value of $n = 3$ is generally adapted, and for sand a value of $n = 8$ has been accepted A committee of the American Society of Civil Engineers have recommended the adaption of the Boussinesq formula (1) for determining the stresses in foundations.

By resolving the equation (2) into 2 components the value of the vertical stress at any point in the supporting media under a concentrated load will be

$$\text{p vertical} = \frac{(n-2)P}{2\pi r^2} \cos^{n-2} \alpha \quad \ldots (3)$$

if $n = 5$, the formula reduces to

$$\text{p vertical} = \frac{3P \cos^3 \alpha}{2\pi r^2} \quad \ldots\ldots\ldots (4)$$

which is similar to equation (1)

Vertical stresses in a homogeneous mass under a concentrated load will present a distribution of stress similar to that shown in Fig. 2. This example of stress distribution has been developed by Goldbeck from experimental results and which are illustrated in Fig. 3

The stress on the axis of a load on a circular area is given by the equation :—

$$p_z = p_0 (1 - \cos^3 \alpha) \quad \ldots\ldots\ldots\ldots (5)$$

where p_z : Intensity of stress at any depth z on the axis of the load

P_0 : Unit load on the surface

α : angle formed by the vertical axis, and a line drawn from the point on this axis where the stress is to be determined to any point on the circumference of the loaded area

The stress on the centre line of a strip load is $p_z \cdot Kp_0 \ldots\ldots$ (h) $\ldots\ldots\ldots (6)$

and

$$K = \frac{1}{\pi}\left(\text{Sin}^{-1} \frac{2\mu}{1+\mu^2} + \frac{2\mu}{1+\mu^2} \right) \quad \ldots\ldots (7)$$

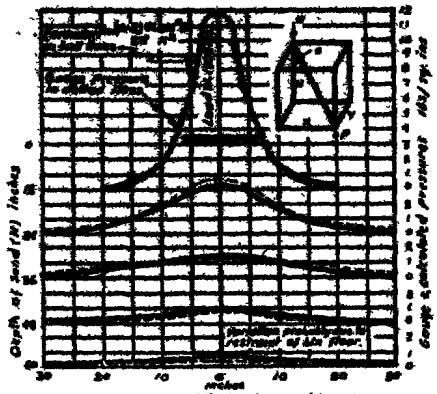
$$\mu = \frac{b}{z} \quad \ldots\ldots\ldots\ldots\ldots (8)$$

where 2b ; width of strip
and z depth at which the stress is to be calculated

Hence if the stress-strain curve for the soil is known it is obvious, from equations (5) and (6) that the settlement under a loaded area is directly proportional to the diameter or width of the area. From this example, it would appear to be necessary to vary the intensity of loading in inverse proportion to the size of the foundation, which is at variance from the traditional design which assumes that the foundations should be designed for a constant intensity of pressure irrespective of the size of the footings, in order to obtain even settlement of the structure.

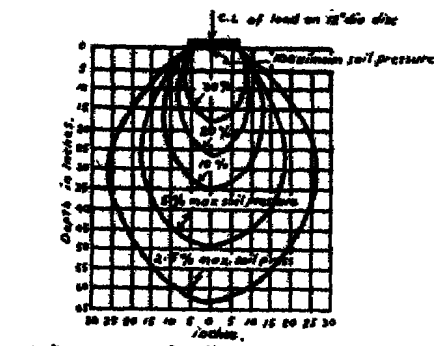
Tests made by Dr. Emperger on cohesive soil to determine the effect of the size and shape of the loaded area on the settlement, at equal loads per unit of area have shown that the settlement increases almost in direct proportion to the area loaded. Other investigators have also proved similarly. However, a number of experiments have shown that the settlements are independent of the loaded areas. To explain these, Professor Terzaghi lays down the following principles :—

(1) The ratio of the diameter of the loaded area to the settlement



U. S. A. Bureau of Public Roads sand bin pressure measurements compared with theoretical formula.

Fig 2.



Bulb of pressure in soil under load based on experimental results reported in Fig 2.

Fig 3.

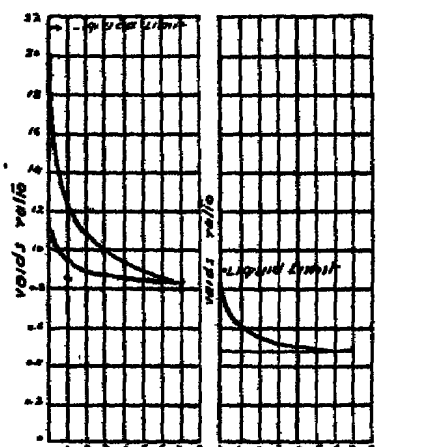
produced by a given load per unit of area depends on the cohesion of the soil In non-cohesive soils, the size of the area has little influence on the settlement, but when the cohesion is considerable the settlement increases in proportion with the diameter of the loaded area.

(2) As the depth of the foundation increases, the settlement produced by a given load per unit area decreases, but the ratio between the settlement $(\Delta)_0$ for a foundation at zero depth and the settlement $(\Delta)_z$ for a foundation at a distance (z) below the surface of the ground depends on the ratio $\frac{z}{d}$ between the depth of foundation and the diameter of the loaded area, and not on the depth (z) alone.

(3) The effect of ratio $\frac{z}{d}$ on the settlement is less as the cohesion increases.

According to these principles, inconsistancies in settlement of structure may be explained as due to the nature of the ground tested and the ratio $\frac{z}{d}$. As an example, if a foundation 5 ft deep reduces the settlement of a 5 ft square footing by 50% compared with the settlement of a footing at ground level, the effect of a footing 10 ft square in 5 ft. deep in reducing settlement will be much less. To reduce the settlement of a footing 10 ft. square by 50%, it would be necessary to make it 10 ft. deep, thus obtaining the same value for $\frac{z}{d}$ in both cases. Settlements are recognised as being of two types, namely, those due mostly to lateral flow with little or no consolidation and those due to a combination of lateral flow and consolidation The former occurs where the permeability of the ground is small, since its contained water then escapes slowly under the pressure and the speed at which the compressed soil decreases in volume is low. Experimental results and observations on Engineering Structures have shown that consolidation of clay as the result of extrusion of water is a slow process

Figure 4 gives the stress-strain curve for two soils in which the loads are plotted against the voids ratios With the curves shown in fig 4 and the distribution, of pressure in the foundation calculated, an idea of the settlement to which a footing would be subjected, may be arrived at However, a complete



Compressibility of soils
Fig 4

solution of this problem is not possible as the effect of cohesion is not taken into consideration in the calculation at all. Settlement of foundations due to shear has not yet been fully developed, as theories connecting it, are at a very controversial stage

Experiments by Kogler and Scheidig at the School of Mines, Freiberg, "(Bautechnik)" showed that the distribution below a stiff circular plate supported on sand is approximately parabolic, the pressure varying from zero at the edge to a maximum at one centre which is as much as 2½ times the average pressure. The influence of the stiffness was found to be considerable when the distribution due to a loaded area of 45 cm diameter placed centrally on a plate 99 cm. in diameter was sought at 40 cm below the centre of the load, was much greater than the average, and decreased to zero near the edges. The maximum pressure varied between 103 per cent and 181 per cent of the mean pressure on the larger plate

According to a report of the Building Research Station experiments have shown that the pressure exerted by sand on the under side of a foundation, placed as it is, not uniformly distributed, but increases towards the centre from a negligible value at the edge, and that the B Ms on raft foundations or similar spread footings are less than is usually supposed

Bearing Value of the Soil, and Settlement of Structures

The minimum unit load, which causes failure of the earth mass, when acting at the base of a structure may be termed "Ultimate bearing power". In all foundation designs, the depth of the base of the structure and the shape and area of the base should be indicated, which is not usually done in the traditional design

We shall next consider the case of a load acted on by a structure on an elastic layer of foundation (fig 5)

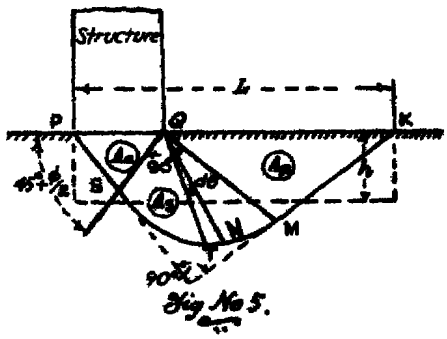


Fig No 5.

It may be seen from the figure PQS is a compressed zone and QSMK is an yielding zone which is usually termed

as plastic zone. In this case the ultimate bearing value of the soil (q) may be expressed (according to Prandtl) in terms of the unit cohesion (c) of the given material and its angle of friction ϕ

$$q = \frac{c}{\tan\phi}\left[\tan^2\left(45^\circ + \frac{\phi}{2}\right) e^{\pi \tan\phi} - 1\right] \quad (9)$$

In the case of many of the cohesive soils, ϕ is negligible and Equation (9) becomes

$$q = 5{\cdot}14\, c \quad \ldots\ldots\ldots\ldots (10)$$

In the study of foundation problems it is generally assumed the prism PQS is an active prism and QMK a passive prism connected with a curve SM, and lines PS and MK being tangent to the curve. The failure line PSMK is thus smooth and has no breaks and so satisfies the requirement of continuity

If the structure lend to tip about edge B and to push out a prism of earth as limited below by the failure line (fig 5), the earth mass yields along radius vectors such as QS, QM, QN radiating from edge B According to Mohr's theory of failure, the failure line SM makes an angle $90^\circ - \phi$ with these radii *i.e.*, curve SM, bounding zone SQM from below passing through the tip of all radii vectors, is a logarithamic spiral

Now let A_1 = area of active prism
A_2 = area of DBG zone
A_p = area of passive prism

For a cohesion-less mass ($c = o$ and $\phi = o$).

Prandtl's formula furnishes $q = o$ This is because one mass below the base of the structure is assumed to have no weight Karl Terzaghi improved Prandtl's results by introducing an average height of the material (h) (fig 5)

$$Lh = A_2 + A_3 + A_p \quad \ldots\ldots (11)$$

Introducing a designation,

$$h\sqrt{\tan\phi} = c' \quad \ldots\ldots\ldots (12)$$

The corrected ultimate bearing value may be computed from the Prandtl's formula (9) by replacing c with $c + c'$.

According to Rankine, the equilibrium of the active prism PQR [figure 6(a)] as

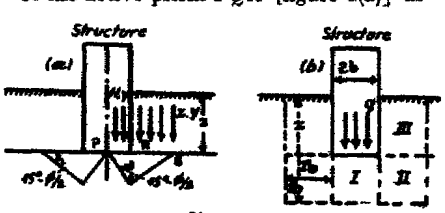


Fig 6.

a result of the load from the structure, tends to move downwards and the passive prism RQS loaded with earth resists the upward movement Rankine suggests the depth for a line foundation, as follows :—

$$z = K_a{}^2 H \quad \ldots\ldots\ldots\ldots (13)$$

where the value K_a is termed *Rankine value* (active Rankine value), and the value of which is given below

$$K_a = \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) = \tan^2\left(45^\circ - \frac{\phi}{2}\right) \quad \ldots\ldots\ldots\ldots (14)$$

and the symbol H denotes the height of the column made of the given earth, which exerts on the base of the structure the same unit pressure $\sqrt{H}$ as the structure itself The ultimate bearing value of the soil would then be

$$q = H\sqrt{} = z\sqrt{K_p{}^2} \quad \ldots\ldots\ldots\ldots (15)$$

where the value K_p is termed as the passive pressure :—

$$K_p = \left(\frac{1 + \sin\phi}{1 - \sin\phi}\right) = \tan^2\left(45^\circ + \frac{\phi}{2}\right) \quad \ldots (16)$$

Terzaghi has improved on Rankine's formula by considering the weight of the mass of the earth below the base of the structure and the shape of the foundation, which have been neglected

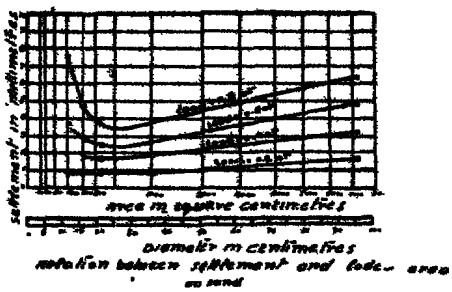


Fig. 7

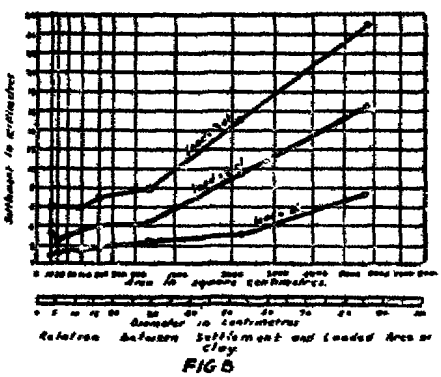


FIG 8

by Rankine, and considered equilibrium of zones I, II, III [figure 6 (b)]. He suggests the formula for ultimate bearing value in the case of a strip foundation of width 2b, as follows .—

$$q = b\sqrt{K_p{}^2}\left[1 + \left(\frac{z}{b}\right) + m\left(\frac{z}{b}\right)^2\right] \quad (17)$$

the value of the constant (m) is generally below $\frac{1}{4}$, so in the case of a shallow foundations where the value of $\frac{z}{b}$ is negligable, the formula (17) reduces to

$$q = b\sqrt{K_p{}^2}\left(1 + \frac{z}{b}\right) = \sqrt{K_p{}^2}\,(b + z) \quad \ldots\ldots\ldots (18)$$

At greater depths the term $\left(\frac{z}{b}\right)^2$ in equation (17) should not be neglected, as it rapidly increases with the depth and gives too large bearing values.

If there is any doubt about the character of the soil or if it is intended to use unit load in excess of those prescribed, the code of practice requires loading tests to be made. This is also because of the lack of knowledge of the exact shearing strains and the vertical components which are induced in the soil on the application of loads on them.

Dr Kogler carried out a number of load tests with loaded areas of general sizes on clay and sand. Figs. 8 and 7 are based on the results obtained from those tests.

From the graphs it is seen that the settlement under load proportionately increases with the diameter of the test area, in both soils. In fig. 8, upto an area of 500 sq. cms. the curves do not prove the above statement, the reason being, due to lateral displacement of the soil under the load *i.e.*, the failure zone II in fig. 6 (b). This effect has the greatest proportional result in the case of the smallest areas under test.

In order to determine the type of foundations and other details for all important structures, subsurface soil conditions should completely be explored. The following factors are of importance in such cases.

1. The variation of the initial stress conditions in the soil, as defined by the natural *i.e.*, pre-consolidation load, as defined by Casagrande. This pre-consolidation load must be known in order to make a reasonable estimate of settlement due to further consolidation

2. The variations of the shearing characteristics of the soil, as defined by the natural shear strength under the pre-consolidation load and by the apparent angle of internal friction. These characteristics influence greatly the stability of foundations.

3. The variation of compressibility of the different soil layers in the deposit. Tapering or lenticular layers of compressible material, will not only cause serious settlement, but may also tilt the structure as a result of unequal Settlement

4 The determination of the rate of settlement as defined by the co-efficient of consolidation

Experiments show that the distribution of the contact pressure of a rigid foundation, on a stiff clay may be expected to be, as shown in Fig 9

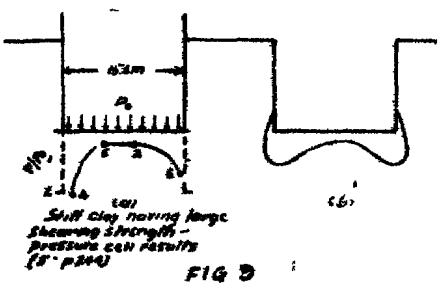


FIG 9

Comings has shown that the effect of different assumed types of contact pressure distribution becomes negligible at a depth of twice the footing width or diameter as indicated in fig. 10. Terzaghi

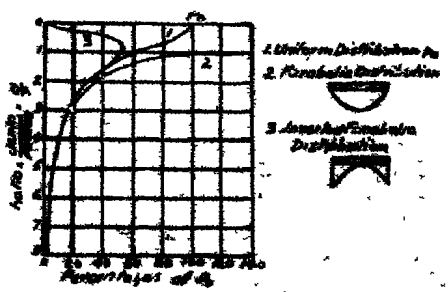


FIG. 10

has pointed out the importance of the distribution of contact pressure on settlement in his "opening Discussion on settlement of structures" (Proceedings Int. Conf. on soil. Mech. and Found Eng. Harvard University, Vol. III, P. 79, 1936), and shows that 0·8 times the settlement takes place within a depth of about 1·5 times the width of the footing, which corresponds to a pressure contour of p as indicated in fig 11, excepting, of course, settlement due to consolidation of deeper soil layers.

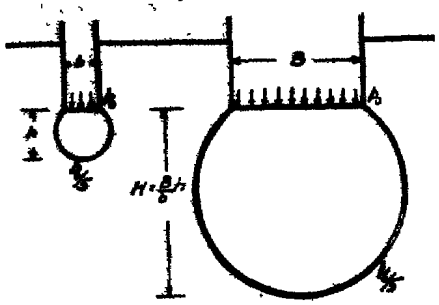
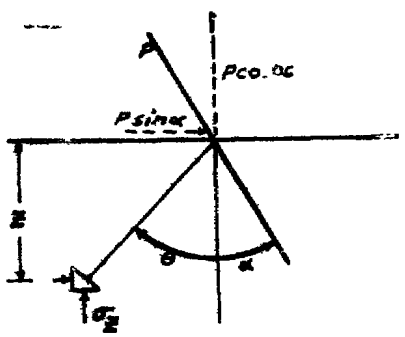


FIG. 11

Eccentric loads and lateral forces produce rotating moments and consequent tilting of the foundations An approximate picture of the distribution of vertical pressure beneath a foundation as a result of tilting can be obtained from the two dimensional cases of the theory of elasticity, by a combination of line loads perpendicular and parallel to the boundary of the semi-infinits solid as the figure 12 and is given by the equation.



$$\sigma_z = \frac{2P}{\pi z} \cos^3\theta \cdot \cos(\theta + \alpha)$$

FIG. 12

Radial pressure

$$\sigma_r = \frac{2p}{\pi r} \cos(\alpha + \theta) \quad \ldots \quad (19)$$

Vertical pressure

$$\sigma_z = \frac{2p}{\pi z} (\cos^3\theta \cos(\theta + \alpha)) \quad \ldots (20)$$

where p is the unit pressure.

As there is concentration of pressure near one edge (fig. 13) due to eccentricity

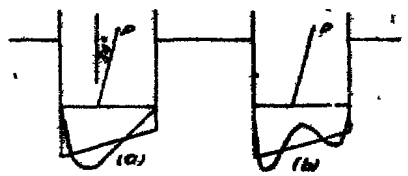
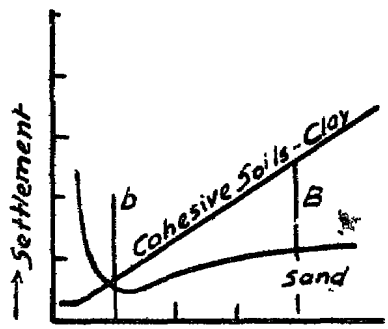


FIG. 13

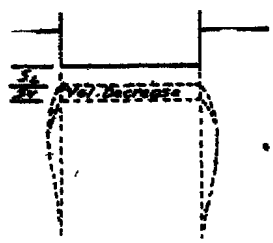
and rotating moments and because 80 per cent of the settlement occur within a depth of about 1·5 times the breadth of the footing, it may be expected that tilting would be greater for the same reason that the settlement of a larger area is greater than a smaller as indicated in figures 11 and 14 and lateral loading tests on piles have shown that quite definite lateral resistance can be counted on, if the unit lateral forces are kept low. If the pressure due to tilting is excessive, lateral yielding of the soil under the load may decrease this stabilising pressure, so that in plastic soil it should not be counted on for stability.



Size of Loaded Area

FIG. 14

Terzaghi has stated that the settlement of a structure founded on clay deposit may be divided into two component parts S_L and S_V respectively as shown in fig. 15 The relation between S_L and



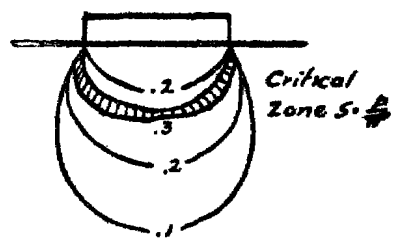
S_L - Settlement due to lateral yielding without volume change, immediately

S_V - Settlement due to compression of soil within the vertical lines, very slowly

FIG 15

S_V is defined by Poissons's ratio μ. As the consolidation proceeds it assumes the real value and the vertical settlement increases to $S = (S_L + S_V)$ Mason has made an important contribution in his correlation of surface loading-tests with unconfined compression tests for cohesive soils and as a result has defined a method for estimating that component of the settlement S_L due to lateral yield from the unconfined compression test Casagrande has stated that the lateral settlement may be computed from the results of the unconfined compression test for S_L and from the consolidation test for S_V In the case of an isotropic material with Poissons' ratio temporarily equal to $\frac{1}{2}$, the strain as defined by $\frac{S_L}{b}$ is 59 per cent of the strain obtained in an unconfined compression test as defined by S_V/B.

A E. Cummings has pointed out that the elastic theory verified by photo elastic studies show that there is a concentration of shear stresses at the edge of a rigid load area, as shown in figure 16, and Prandtl has shown that



Shear Stress in Critical Zone $S \cdot \frac{b}{\pi}$

Plastic State - $b = \pi C$

Rupture - $b > (\pi + 2) C$

FIG. 16

the plastic state begins at the critical point under the edge when the vertical pressure p becomes $\pi \times c$ where c is the cohesion or natural shear strength. The condition for failure occurs when p exceeds c $(\pi + 2)$ at the edge

Finally, the settlement due to consolidation of the underlying soil is computed, having determined the distribution of vertical pressure and applying the usual method of settlement computations.

The most suitable position for the foundations of a structure cannot always be defined by borings, trial pits and probings. The execution of such investigations causes often great difficulties and considerable expences.

For the planning of foundations, geophysical methods are necessary, particularly for finding the best position in extended areas, and to locate the positions of water-logged soils, filling and other unstable strata lying above bearing rock.

The methods available still require to be checked by borings Borings give exact, but only local information whilst geophysical methods allow to investigate the soil conditions of large areas

There are primarily two methods for investigating the soil conditions of large area, *i e*, the electric and seismic process The first method is based on the different electric resistance of different soils. To measure these resistances mostly a 4-pole arrangement is used. The second method, seismic tests, operates on the

CONCRETE FOOTINGS ANCHOR STEEL TOWERS

CAST-IN-PLACE Concrete Footings have been successfully used by the Southern California Edison Co. to anchor steel towers supporting its third transmission line from Boulder Dam to Los Angeles Their use reduces the high cost, time and manpower required to dig the 5-ft. square holes required for steel footings used on previous lines, as well as providing a more permanent footing by eliminating corrosion of the buried steel.

Installation of the concrete footings is simple A hole 18 to 36 in in diameter is drilled by a power auger to a depth of 9 to 15 ft. The width and depth depend upon the ground formation and type of tower to be erected. Water soaking of the hole site prevents cave-ins. Next a steel reinforcing cage, 6 in. smaller diameter than the hole size, is centered in the hole and the stub angle of the tower is accurately positioned and braced in the hole. Surveying instruments are used in setting the angles to insure the required 1/100-ft. tolerance.

Concrete is poured by gravity down a jointed trough from the mixer to the hole form, which lines the footing for the upper 2½ ft. only. Wetting of the earth surrounding the newly-poured concrete and use of curing compound on the exposed surfaces prevent too-rapid curing. —(*With acknowledgments to "Construction Methods"*)



POWER-DRIVEN AUGER drills hole for cast-in-place concrete footing for steel towers.



CONCRETE IS POURED via trough from mixer to hole, which is kept moistened after pour is finished to avoid too rapid curing.



BOULDER DAM third transmission line towers are carried on permanent concrete pedestal footings poured in drilled holes. Stub base angle is cast into concrete.

(*Continued from previous page*)

principle of reflection and refraction The latter is chiefly used for geological investigations of foundation sites (measurements for low depth). The explosion of a dynamite charge causes vibration in the ground The propagation of waves is recorded by seismographs and oscilographs. The nature and thickness of strata can be examined also by dromochromatic curves. Example: investigation carried out in Department Meurthe et Moselle, France

Conclusion

In Soil research, the magnitude of the volume change due to increased pressure, the permeability and the cohesion or the shearing resistance are sufficient data for their investigations. Tests made in Sweden have shown, that disturbing a soil without changing its volume of voids and its water-contents may reduce its cohesion by an amount ranging from 75 per cent to 96 per cent of its original value. Hence the importance of obtaining a sample in its undisturbed state. Secondly, the ground varies in character in accordance with the manner in which the deposit was originally formed, thus necessitating a very large number of tests before a fair average can be struck.

It should not be assumed that the deviations which have been suggested in this paper from the traditional design of foundations has met with general approval of all engineers who have specialised in this problem On the contrary it has been severely criticised by engineers in practice. Unless a theory can be developed which will give the safe load on any particular kind of ground, the engineer will obtain little assistance from the most complete knowledge of permeability, compressibility, etc However, the selection of foundation by a method which approaches working conditions is one which can be usefully developed.

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DESIGN OF WILBUR CROSS PARKWAY BRIDGES

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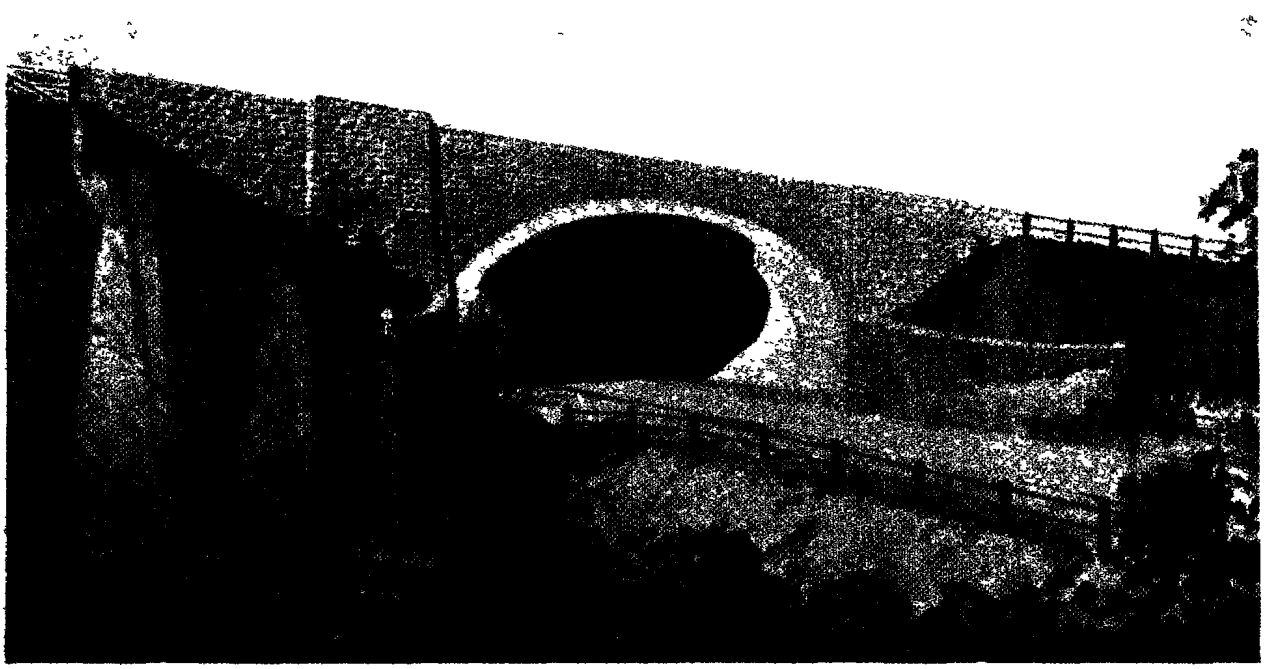


Fig. 1.—Wingwalls of the arch bridge over Fountain St. are carried on columns, as indicated in phantom form at the left.

CONTENTS IN BRIEF—*Proposed for the Wilbur Cross Parkway—Connecticut's superhighway for speeding traffic between New York City and Boston—are several noteworthy grade-elimination structures Fountain St Bridge is a skewed reinforced-concrete arch with wingwalls supported on columns, the crossing over Route 69 is carried on a two-span welded, continuous girder, each span of which is 125 ft, and Dixwell Ave. Bridge is an all-welded 126-ft deck plate-girder span designed for composite action with the concrete slab Major considerations affecting the design of these and other structures are outlined*

CROSSING CONNECTICUT on a diagonal, from the New York State line at the southwest corner to the Massachusetts line at the northeast (Fig. 2), a superhighway is being constructed for through traffic between New York, New Haven, Hartford and Boston In the vicinity of New Haven, on a section called Wilbur Cross Parkway, grade crossings will be eliminated through the use of several interesting bridges These include reinforced concrete frames and arches, welded continuous girders, and structures designed for composite action between the concrete roadway slab and the steel members

In selecting the type of bridge to be used many factors, varying with each location, had to be taken into consideration Among them are grade, alignment and clearance requirements of the intersecting roads, topography, foundation conditions and scenery. However, before the effect of these factors on design are discussed, it might be well to examine the nature of the proposed superhighway.

Highway divided into Sections

The western end of the parkway, an extension of the Hutchinson River Parkway in New York State, has been completed to a highway intersection west of New Haven (Fig 2) The section from the New York line to the Housatonic River is known as the Merritt Parkway (*ENR* July 20, 1939, vol p 70)

Between the Housatonic River and the Connecticut River is the Wilbur Cross Parkway (*ENR* Nov 6, 1941, vol p 640), a dual-lane limited access road, traffic on which is restricted to pleasure vehicles. The remaining section, known as the Wilbur Cross Highway, is identical in construction but has no traffic or access restrictions. Completed portions of these sections are indicated in Fig 2 by solid lines

In cross section, the Wilbur Cross Parkway differs but little from the Merritt Parkway. Total width of each lane from the edge of the centre mall to the guard railing, which includes the paved shoulder, is in most cases 35 ft, with widening where necessary for ingress and egress The central mall varies from 10 ft to the 30 ft required at the entrance to the tunnel north of New Haven Loading to be carried is H20-S16

With regard to grade separation structures, a somewhat different design policy from that used on the Merritt Parkway now prevails On that highway, many of the overpassing structures are single-span rigid frames of concrete, where span length permits While this type of structure is economical and well adapted to grade separations, overuse may be monotonous

Because single-span bridges do not allow as much freedom in architectural treatment as do multiple spans, all structures over the Wilbur Cross Parkway are designed as two-span continuous bridges where foundation conditions permit If boring data indicate existence of dubious foundation conditions, either simple spans or individual structures are specified.

Steel will be used to a much greater extent on the Wilbur Cross Parkway

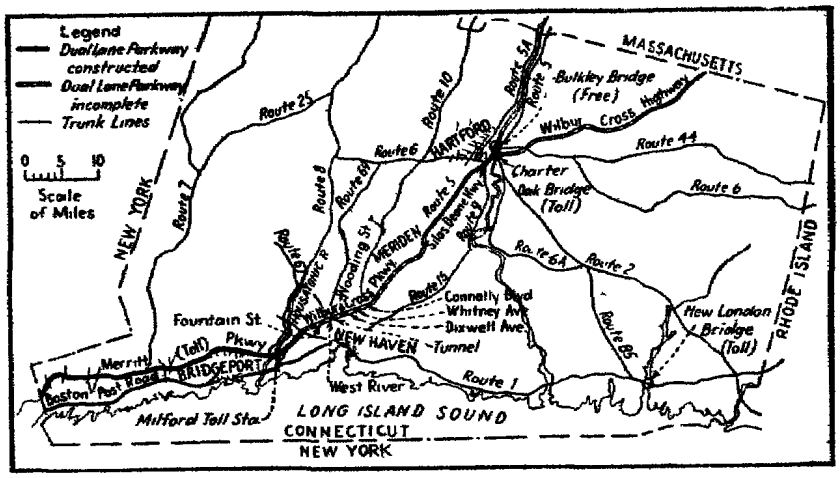


Fig 2.—Connecticut's express highway, which when completed will cut diagonally across the state, is divided into three sections, Merritt Parkway, Wilbur Cross Parkway and Wilbur Cross Highway. The two parkways are limited access routes on which no trucks are allowed.

than on the Merritt Parkway In some cases, length of span makes concrete uneconomical, but in a number of the shorter span structures, particularly where uncertain foundation conditions exist, rolled-beam superstructures will be used with no sacrifice of pleasing appearance Also, since a complete "face lifting" may easily be obtained by sandblasting and repainting steel with little doubt of the results, ease and economy of maintenance favour steel construction

Grade of Fountain St. a problem

The treatment of several road intersections in the vicinity of New Haven, where extraordinary topographic conditions at the site have imposed difficult conditions, is worthy of note Foremost among these crossings is the bridge carrying the parkway on a 6 per cent. grade over Fountain St, which at the intersection has a grade of about 10 per cent. and is on a moderately high fill

On either side of Fountain St, the terrain rises to a height of sufficient extent to justify a parkway overpass Had Fountain St been carried over the parkway, a cut of several hundred thousand cubic yards might have been necessary Comparative designs favoured the structure selected, even though a bridge over the parkway would have been considerably simpler in design and consequently less expensive, and a large portion of the excavated material, predominantly rock and boulders, could have been used advantageously elsewhere

Several features in the design of this structure are outstandingly different from any other on this or the Merritt Parkway The bridge is a single span, reinforced concrete arch, skewed at an angle of about 15 deg 40 min Because of the 10 per cent grade on Fountain St and the usual necessity of holding down the grade of the overpassing road the frame was designed as three separate longitudinal sections In each of the sections, the skewed distance between faces of abutments is 60 ft, but the rise relative to the height of leg varies to conform to the street grade

Designed for H20-S16 loading, the arch rib varies in thickness from 2 ft. at the crown to 3 ft 1⅛ in at the springing line From this last section the legs are widened on the roadway side on a slope of ⅞ in in 12 in for a distance varying from 7 ft. 6 in to 10 ft. and then are brought sharply back to the springing line width Since the parkway has a 6 per cent. grade at the intersection, the height of spandrel walls is different at each abutment of the bridge.

Columns carry wingwalls

The east wingwalls, which are on the side of the high embankment, are supported on columns between which the fill is permitted to pass. Near the toe of the slope, the embankment is retained by a low wall (Fig 1 and 5) This type of construction should afford an appreciable saving of materials and reduce costs.

The exposed surface of the Fountain St. bridge will be faced with random ashlar, except that granite dimension stone will be used for the arch rings and wherever a sharp outline is desirable (It should be noted that the specification of the Connecticut State Highway Department for random ashlar masonry facing does not require all stones to be square or rectangular but "shall simulate the appearance and conform to the dimensions shown on the plans.") Because of the picturesque setting, the expenditure for natural stone facing, which will be no small part of the total cost, is considered well justified.

Rolled beams used for some bridges

Near Fountain St is the broad, level flood plain of the West River, Wilbur Cross Parkway is carried over Route 67. Foundation conditions at this site are such that light settlement could occur, a condition that usually precludes the use of frames and arches

The type of bridge selected in typical of many others used under similar geographical conditions The superstructure consists of rolled, wide-flange I-beams, with special provision made



Fig. 3.—This structure over Route 67, typical of many others used under similar geographic conditions, consists of rolled steel beams designed for composite action with roadway slab. Abutments faced with precast slabs of artificial stone.



Fig. 4.—Welded continuous girders carry the parkway over West River and Route 69. Details are shown in Fig 5

to tie the concrete floor slab to the top flanges to develop horizontal shear Composite construction of this type has proved advantageous where headroom is tight. It reduces deflections and is economical.

Since an ornate structure would not be appropriate in this region, the architectural treatment is simple and intended to be in harmony with the surrounding country Exposed portions of wingwalls, abutment faces and parapets are overlaid with precast concrete slabs of artificial stone This precast slab facing was used on the Merritt Parkway with entirely satisfactory results, both esthetically and economically, and has the advantage that the large slabs, 12 to 20 sq ft in area, replace form work and eliminate the necessity of finishing the surfaces

While a wide variety of colours and patterns are available, only cast stone simulating gray and pink granite and brown, white or red sandstone will, in general, be used for bridge ornamentation However, in a few cases a micaceous element will be added to produce a scintillating effect under headlights or the direct rays of the sun Although this sparkling stone has a pleasing appearance at first, it is somewhat susceptible to dust and loses most of its brilliance after a few years To restore the original texture, it may be necessary to resort to washing under pressure or air blasting

Surprisingly, a different type of facing is specified for a structure carrying the parkway over Route 69 and West River only about 1,000 ft easterly from the Route 67 bridge and in the same valley. In consideration of the river crossing and because the site is on the approach to West Rock Mountain and a tunnel, granite facing was specified for the pier, wingwalls and abutments (Fig. 4)

Continuous girder over Route 69

At this site the parkway will be carried over Route 69 and West River on eight two-span welded continuous girders with variable depth of section (Fig 5). Each span is 125 ft. between bearings

Symmetrical about the centre support, the girders are 4 ft deep at the centre of each span, 6 ft at the free ends and 9 ft 6 in. at the centre bearing, measured between inner faces of flanges The web is ½ in thick, except for a length of about 28 ft over the middle support where a ⅝-in plate is used Top and bottom flange plates are 20 in wide and vary in thickness from 1 to 1¾ in , as indicated in Fig 5 The parabolic lower flange rises 2 ft at midspan relative to the free end and 5 ft 6 in relative to the intermediate support

Before this design was finally decided upon, two separate, detached structures were considered. However, it was found that these would increase the cost by over 40 per cent Another disadvantage of individual bridges is that a bad traffic hazard—a blind intersection—would incidentally be created where a local road enters Route 69 close to the proposed crossing

Furthermore genuine advantages accrue from the adoption of the continuous type, particularly at this site Reduction of positive bending moment due to continuity permits a decrease in depth of girder at the centre, which in turn allows increased headroom This is important because in order to maintain the necessary 14-ft vertical clearance, it would otherwise have been necessary to raise the grade for a considerable distance in both directions at the expense of a greatly increased borrow item and also higher and costlier wingwalls

The east abutment, east wingwalls and centre pier will be supported on 30-ft steel piles driven to rock Unless unforeseen foundation conditions should be uncovered, the probabilities are that piles will not be required for the west abutment and wings.

From the architect's viewpoint there seems to be little, if any, argument against welded structures with smooth, flowing lines easily assimilated by the eye, as compared to the ugly, mechanical appearance of riveted girders. In addition, riveted construction in this case was found to be more expensive by over 11 per cent , using estimated prices of 7c. per lb for welded against 6c for riveted steel in place complete. Weight of the welded steel was estimated at 780,000 lb , while the same superstructure assembled by riveting would weigh about 1,028,300 lb Thus :

1,028,300 lb at 6c = $61,698 riveted

780,000 lb at 7c = $54,600 welded

$ 7,098 saved

Estimated cost of the bridge is about $200,000 or $11.27 per sq ft of projected floor area, based on the average level of pre-war unit price bids What actual bid prices will be is still a highly debatable question

Design of unsymmetrical frames

Another two-span overpass of interest is the rigid-frame structure carrying Wooding Street, Hamden, over the parkway on a skew of 13 deg 44 min (Fig 6) The setting is rustic, and an architectural problem was posed by the 10 per cent grade of Wooding Street, which precluded symmetry, especially with regard to the spandrel walls A satisfactory solution seems to have been attained (Fig 5) by lengthening the span on the high side by 5 ft and lowering the rib of each span on the side of the descending grade 1 ft relative to the opposite side

Although a two-span structure, this bridge is not of the continuous type. Rather the two frames are independent, and expansion joints are provided to take care of unequal settlement

The skew span at springing line of the frame on the high side is 46 ft 2 in At the crown the thickness of rib is 1 ft 9 in , increasing to 4 ft at springing, and the legs taper to a width of 2 ft at top of footing Spanning 41 ft 2 in., the other frame is 1 ft 4 in thick at the crown and 3 ft at springing While the footings of each frame are on the same level, those on the longer-span side are 1 12 ft higher

Long-span welded girdars

Farther east, the parkway is carried over two main arteries, Whitney and Dixwell avenues, by deck plate-girder structures differing from each other only in detail Broad streets crossed on a skew required span lengths some-

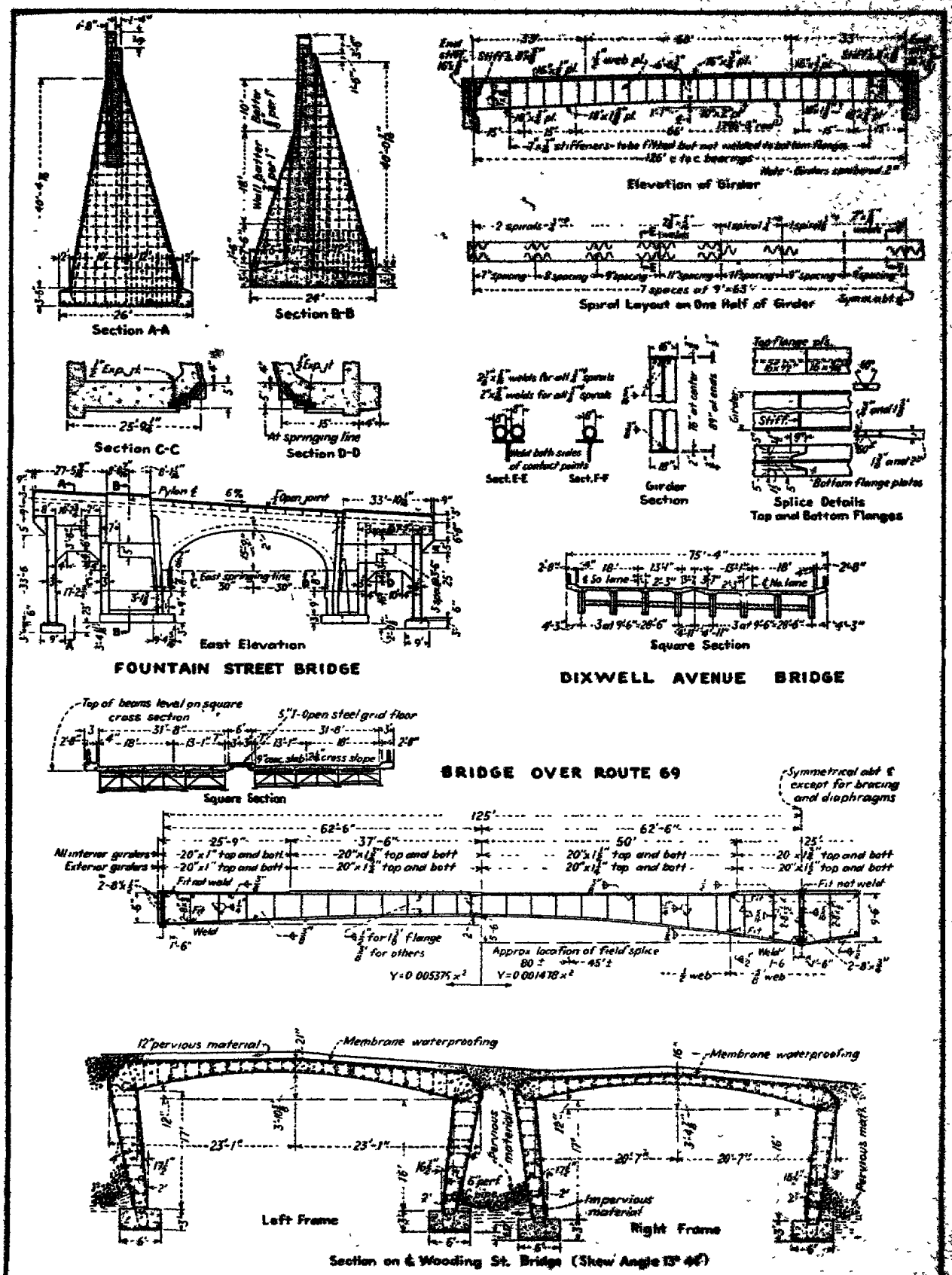


Fig. 5.—Outstanding details of Wilbur Cross Parkway bridges, include design of columns to support wingwalls, welding of continuous girder, proportioning of unsymmetrical skewed two-span rigid frame and design of a long-span welded girder for composite action.



Fig. 6.—The Wooding St. crossing posed a problem in architectural treatment because of the steep grade of the overhead road.

what in excess of those encountered elsewhere on this project.

A satisfactory solution to the longspan problem in both cases would be conversion of both avenues into dual highways for some distance from the bridges, construction of a pier in the separating mall and substatution of a two-span structure. This plan is a good safety measure and sound engineering, but the total cost would exceed that of the design selected

The longer crossing of the two is the 126-ft one over Dixwell Ave., spanned by all-welded deck plate girders, designed as composite beams and having curved lower flanges (Fig. 5) Over Whitney Ave is a similar structure, excepting that the span is 118 ft. 6 in and the top and bottom flange plates are parallel In both cases headroom was limited and a shallow superstructure highly desirable in order to maintain the required 14-ft. vertical clearance.

The girders over Dixwell Ave have a ½-in thick web and vary in depth from 6 ft 6¾ in at the centre to 7 ft. 6¼ in at the ends Top flange plates are 16 in. wide, from ½ to ¾ in. thick, and bottom plates 18 in. wide, from ¾ to 2 in. thick, as shown in Fig. 5 The lighter top flange is due to participation of roadway slab in carrying compression stresses. To insure composite action between the concrete slab and the girders, patented spiral shear connectors are welded to the top flanges.

Welding was specified for the fabrication of these girders because of economic and architectural advantages over riveting, based on experience with ten all-welded bridges now in service in Connecticut. A systematic inspection of these structures, conducted over a period of from one to five years after their completion, revealed that no defects in welding had developed, nor was any permanent distortion visible.

Viaduct spans road and railway

East of these girder bridges is a relatively more complicated intersection. The Northampton Division of the New York, New Haven & Hartford Railroad is carried over Connolly Boulevard, and



Fig. 7.—Parkway is carried over Connolly Blvd. and railroad on 5-span deck plate-girder viaduct on a 4-deg. horizontal curve and 600-ft. vertical curve. About 570 ft. long, the structure carries two lanes of 31-ft. width and is supported on concrete bents, chosen instead of steel for the sake of appearance.

the parkway passes over both, (Fig. 7). A 5-span deck plate-girder viaduct, measuring 570.15 ft. along the centre line, is specified The span at the west end bridges the railroad and the east end spans Connolly Boulevard

Alignment of the highway imposes a few annoying details The viaduct is on a 4-deg. horizontal curve, and each 31-ft. 1-in. wide traffic lane is banked 23.31 in., or ¾ in per foot, while the centre mall is horizontal radially. Furthermore, the structure is on a 600-ft vertical curve with tangent grades of +0.95 and —3.49 per cent.

Piers designed as concrete bents

The horizontal curvature, grade and superelevation leaves little leeway in architectural treatment of the superstructure. However, it was recognized that due attention must be given to the appearance of the piers Several attempts were made to design steel columns on concrete bases, but while requirements of safety and economy were satisfied, those of appearance were not Finally, concrete column bents were decided upon. The cost will be but slightly more than that of the steel columns considered, while the results in appearance will be far more gratifying.

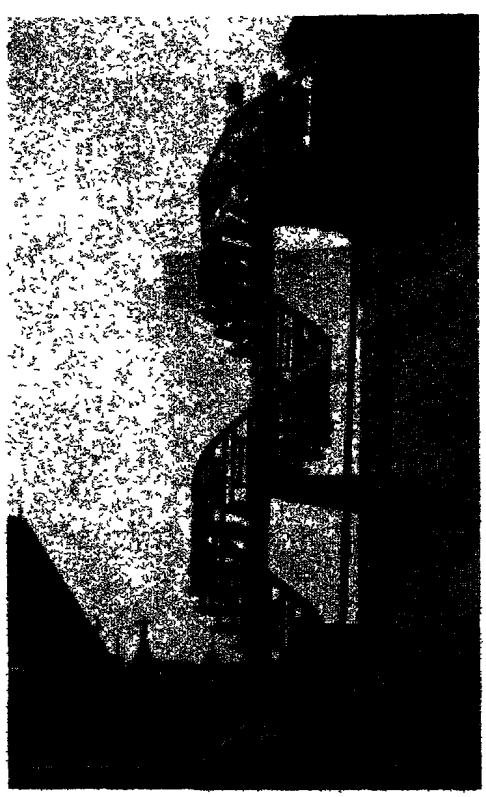
As for the cost of this viaduct and other bridges on the Wilbur Cross Parkway and Wilbur Cross Highway, there may be a wide differential between the actual costs and the estimated, because of the huge total quantities, coupled with the great uncertainties relative to unit bid prices when normal construction is resumed On the basis of 1940 and 1941 *bids*, the estimated cost of the parkway, exclusive of the tunnel, is about $10,000,000, of which nearly $3,000,000 will be expended for bridges of all types, including the necessary approach work, clover-leaves and other connections

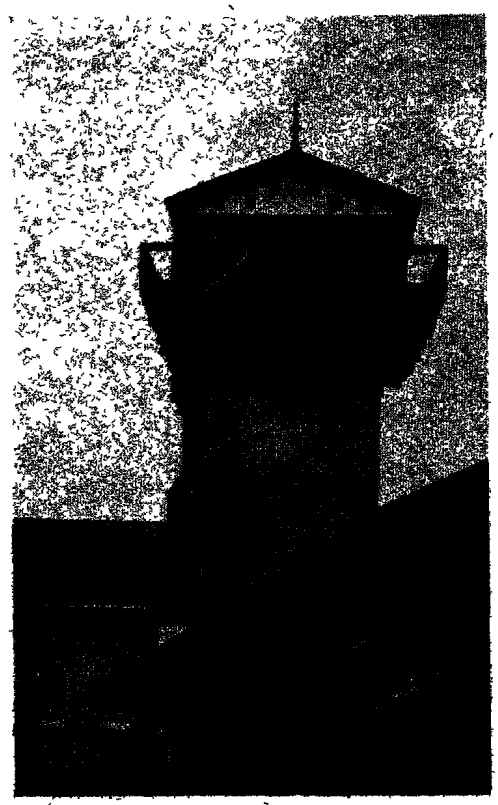
Proposed financing

The project will be financed by tolls collected at toll stations on the Merritt Parkway and by 5 per cent. of the gross highway fund set aside for that purpose. If sufficient money is not thus obtained, a bond issue not to exceed $4,000,000 is authorized by the general assembly.

Owing to the magnitude of the work, it is likely that the parkway and tunnel will not be completed and opened to traffic until at least the fifth post-war year. Since many other projects in Connecticut may be under contract concurrently with this one and since nearby states have an extensive post-war programme that will fully engage their contractors, even if the entire sum necessary for construction were available, there is not a sufficient number of qualified contractors in or near Connecticut to do the work in a shorter time.

The renderings of the grade-elimination structures were executed by George L. Dunkelberger, highway architect. In preparing the designs of the bridges, details shown on the renderings were incorporated into the working plans. This procedure was followed on the Merritt Parkway project, and the photographs of the completed structures reveal a startlingly close resemblance to the pre-construction renderings.—(*With acknowledgments to "Engineering News-Record."*)





TWO INTERESTING PHOTOGRAPHS OF A PRECAST SPIRAL STAIRCASE CONSTRUCTED IN BOMBAY

The staircase starts from about 60 ft. above the ground level.

INGENIOUS FOOTINGS FOR INDUSTRIAL BUILDING

By H. A. SAURBREY, Chief Construction Engineer,
Westinghouse Electric Corporation, Pittsburgh, Pa.

UNUSUAL soil conditions enabled the Westinghouse Electric Corporation to save the entire cost of the form work for several hundred concrete footing installed in connection with one of their new factories. The soil was a fine dense clay which could be carved almost like cheese and would stand up for a long time without support Figure 1 shows how this was taken advantage of in constructing the footings; the excavation was carried down with straight vertical sides.

The surface of the site was practically level over the 100 or more acres on which the buildings were to be constructed Approximately 10 ft. below the surface was encountered a continuous stratum of hard shale, which shale formed an excellent support for the concrete footings These were carried down to the hard shale, penetrating through some 3" of softer shale, which appears in Figure 1 as a dark line separating the hard shale from the clay.

The photograph, Figure 1, shows how the excavation was started In order to guide the excavators and also to protect the top of the excavation from fraying, a simple template composed of 3×12 timber was laid on the surface of the ground as shown This template also formed the foundation for the supports for the anchor bolts, these were hung from framework as shown in photograph, Figure 2. The concrete footing was then poured by the simple expediency of filling the entire hole full of concrete, the completed footing is shown in photograph, Figure 3

The completed factory comprises several groups of buildings erected at different times On the first group of buildings, some 600 footings were placed in this manner without any appreciable loss from the exposed banks of the excavation, it is estimated that a total of not more than 10 or 15 cubic yards of clay were lost. This loss was caused mostly by alternate freezing and thawing of the top of the exposed excavations Of course, any small amount of caved material was carefully removed before concreting, the appearance of the holes in Figure 1 is typical of all excavations Subsequent additions to the same plant encountered somewhat less favourable soil conditions, which however were easily overcome by the introduction of an intermediate set of braces half-way down the excavated hole. The scheme indicated was carried out substantially as shown on approximately 900 footing holes at a saving of form work estimated to be in excess of $60,000 To this figure must be added the saving obtained by dispensing with all backfill The excavation work was partly done by ordinary hand spades which cut the walls of the outside perimeter of the hole straight and square, the loosened clay was thrown into the middle of the foundation where it was lifted out, together with the bulk of the excavated earth, by means of clamshell buckets The bids received on this type of excavation varied somewhat, on the first operation the work was done for .99 per cubic yard, on subsequent additions, the price was higher, the lower bid quoting $1.65 per cubic yard for this type of excavation These prices are based upon the use of union labour throughout

A close inspection of the excavated bank in Figure 1 shows a number of round holes and a couple of pockets, these are the work of the crawfish which abound in the territory These little animals are approximately the same size and appearance as the crawfish commonly found in our streams, but live here in entirely dry soil in burrows underground During a heavy rainfall, the chamber at the bottom of the shaft runs full of water, and serves the crawfish with a moist chamber in which to live. No particular difficulties were encountered because of these burrowing animals, except that the excavators sometimes received an unexpected shower-bath when the wall of the little water chamber burst.

Mr. George Parkman is Director of the Building Construction and Maintenance Department of Westinghouse Electric Corporation. The design and construction was done under the direct supervision of Mr. H. A. Saurbrey, Chief Construction Engineer; Mr. Henry J. Larsen, principal assistant.



Fig 1

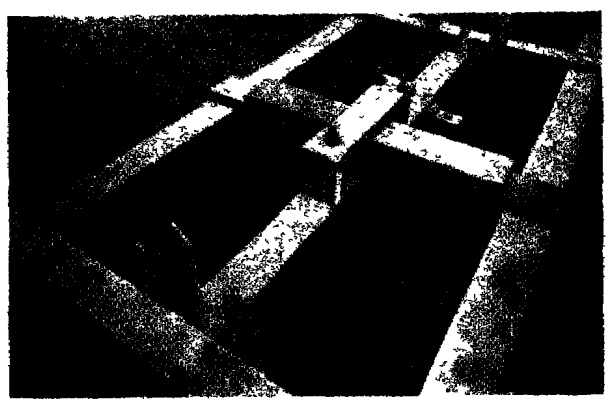


Fig 2.



Fig. 3.

PRE-CAST CONCRETE FLOOR UNITS

By W. SCOTT WILSON

NORMALLY the cost of formwork for reinforced concrete structures is considerable, but with present-day cost and scarcity of timber, the whole question of formwork for in situ concrete work becomes a problem. If we can eliminate formwork on the site and reduce its use to a mere fraction in the production of pre-cast concrete units, we have gone some way, at least, towards part of the solution. For the reason that pre-cast work does not lend itself easily to monolithic constructions, that is, continuous slabs, beams and

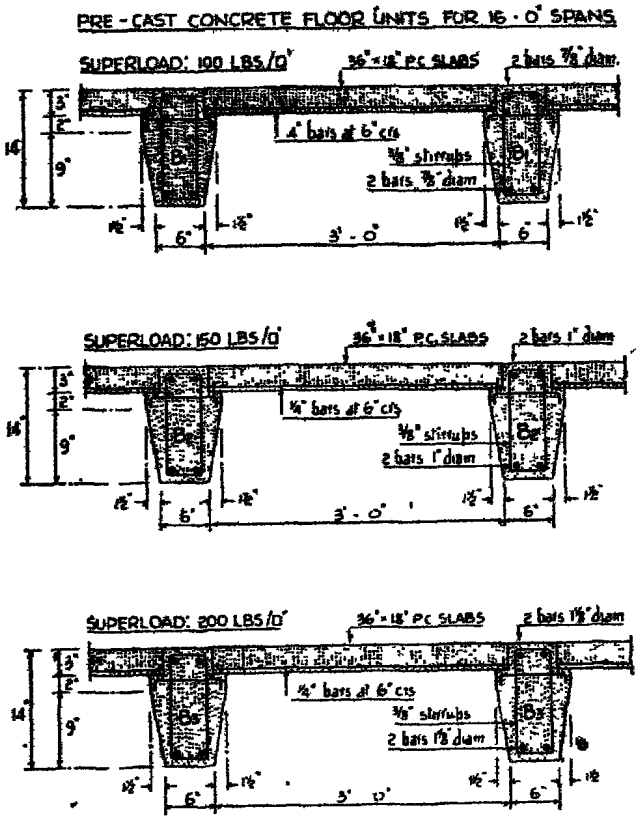


Fig. 1.

columns, its uses are limited. But for single-span floors and roofs, supported upon brick walls, pre-cast units are efficient and economical for a variety of workshop buildings.

All units for floors consist essentially of light slabs that transfer their load to heavier units, or beams, bearing upon supports. Such construction is shown on the accompanying drawings which give the details for spans of 16 ft. and 20 ft. and for superloads of 100, 150 and 200 lb. per sq. ft. The floor units are 36 in. by 18 in. by 3 in. thick, reinforced with ¼ in. diam. mild steel bars at 6 in. centres and ¼ in. diam. bars at 12 in. centres transversely. These units weigh about 160 lb. each and can be handled comfortably by two men. The beams are 14 in. by 6 in. for 16 ft. spans, and 16 in. by 6 in. for 20 ft. spans, and have continuous ledges at each side upon which the floor units take their bearing. After the beams are fixed in position at 3 ft. 6 in. centres, the floor units are laid on and the joints grouted up. The floor is then finished in accordance with the purpose for which it is to be used.

Pre-cast slabs. 36 *in.* × 18 *in.* × 3 *in.*, *span* 3 *ft.* 0 *in.*

dead load	= 3·0 × 1·0 × 0·25 × 140	= 105 lb. per ft. width.
live load	= 3·0 × 1·0 × 200 lb.	= 600 ,,
	total load	= 705 ,,
moment M	= 705 × 36/8	= 3,170 in. lb.
steel area As	= 3170/18,000 × 2	= 0·09 sq. in.
and ratio r	= 0·09/12 × 2·25	= 0·0033

Hence, slabs reinforced with ¼ in. diam. bars at 6 in. centres longitudinally and ¼ in. diam. bars at 12 in. centres transversely are suitable for all superloads up to 200 lb. per sq. ft., the compression on the concrete being around 400 lb. per sq. in. The concrete mix for these slabs is 1-2-4, the maximum sized aggregate being ⅜ in.

Beam Units B1, section 14 *in.* × 6 *in.*, *span* 16 *ft.* 0 *in.*

dead load beam	= 16 ft. × 1·17 × 0·6 × 140	= 1,570 lb.
dead load slab	= 16 ft. × 105	= 1,680 lb.
live load	= 16 ft. × 3·5 × 100	= 5,600 lb.
	total load	= 8,850 lb. = 4 ton
moment M	= 4 × 16/8	= 8 ft. ton
steel area As	= 8/8 × 0·83	= 1·2 sq. in.
total shear S	= 8,850/2	= 4,420 lb.
unit shear s	= 4,420/10 × 6	= 73 lb. per sq. in.

and no shear steel is required.

Reinforcement: At top and bottom 2 bars ⅜ in. = 1·2 sq. in. and 2 sections ¼ in diam., stirrups at 12 in. centres throughout.

Beam Units B2, section 14 *in.* × 6 *in.*, *span* 16 *ft.* 0 *in.*

dead load beam and slabs		= 3,250 lb.
live load	= 16 ft. × 3·5 × 150	= 8,400 lb.
	total load	= 11,650 lb. = 5·2 ton
moment M	= 5·2 × 16/8	= 10·4 ft. ton
steel area As	= 10·4/8 × 0·83	= 1·56 sq. in.
total shear S	= 11,650/2	= 5,820 lb.
unit shear s	= 5,820/10 × 6	= 97 lb. per sq in

value of concrete rib in shear :—

75 × 10 × 6 = 4,500 lb.

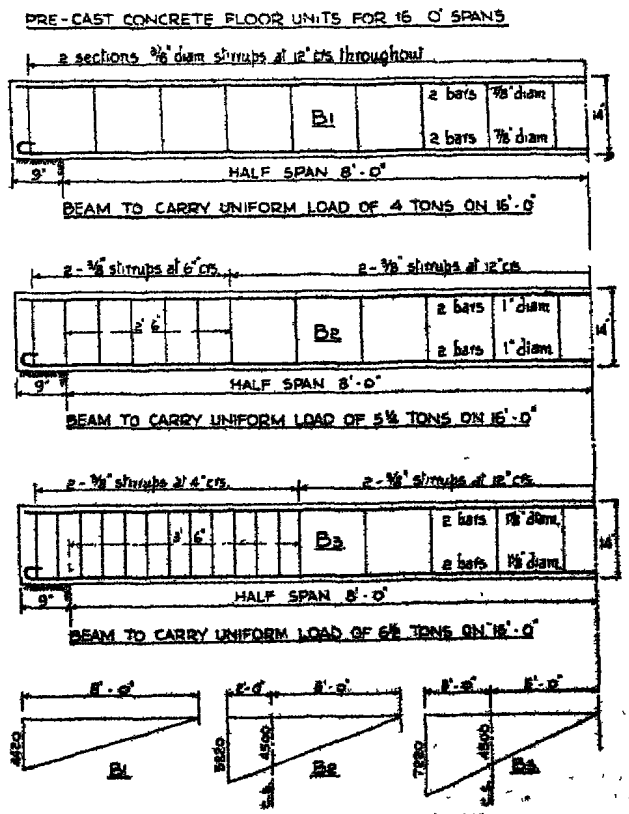
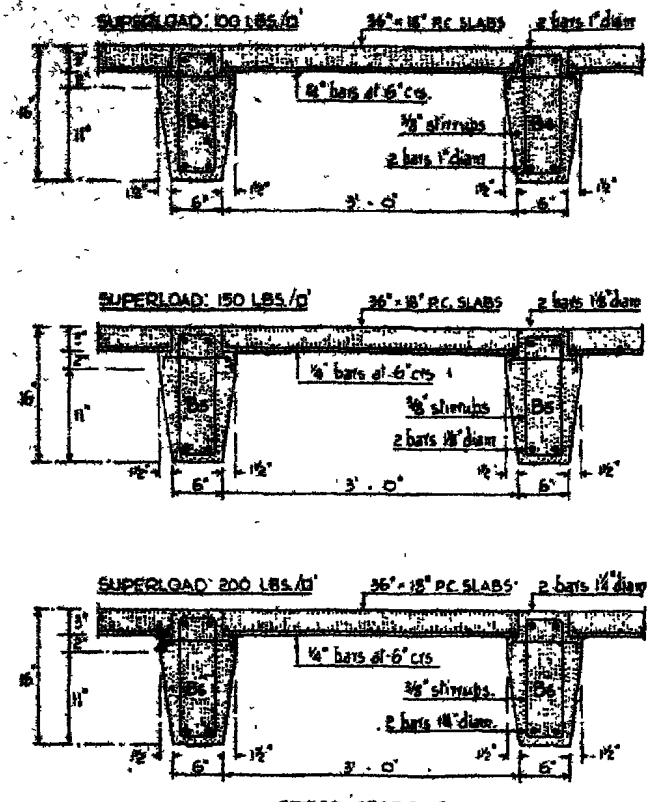


Fig. 2.

distance of critical section from mid-span :—
4,500 × 8/5,820 = 6 ft.
shear resistance
provided = 0·22 × 18,000 × 10/6 = 6,600 lb.
Reinforcement : At top and bottom 2 bars
1 in. = 1,57 sq. in.
in shear : 2 sections ⅜ in. diam. stirrups at 6 in. and 12 in. centres.

Beam Units B3, section 14 in. × 6 in., span 16 ft. 6 in.
dead load beam and slabs = 3,250 lb.

PRE-CAST CONCRETE FLOOR UNITS FOR 20'-0" SPANS.



CROSS SECTIONS.

FIG 3

live load =16 ft. × 3·5 × 200 =11,200 lb.
total load =14,450 lb. =6·5 ton
moment M =6·5 × 16/8 =13 ft. ton
steel area As =13/8 × 0·83 =2 sq. in.
total shear S =14,450/2 = 7,220 lb.
unit shear s = 7,220/10 × 6 =120 lb. per sq in.
value of concrete rib in shear :—
75 × 10 × 6 = 4,500 lb.
distance of critical section from mid span :—
4,500 × 8/7,220 = 5 ft.
shear resistance provided 0·22 × 18,000 × 10/4 = 9,900 lb.
Reinforcement : At top and bottom 2 bars 1¼ in. = 2 sq. in.
in shear : 2 sections ⅜ in. diam. stirrups at 4 in. and 12 in. centres.

Constructional details for 16 ft. spans are shown on Figs. 1 and 2.

Beam Units B4, section 16 in. × 6 in., span 20 ft. 0 in.
dead load beam =20 ft. × 1·33 × 0·6 × 140 =2,230 lb.
dead load slabs =20 ft. × 105 =2,100 lb.
live load =20 ft. × 3·5 × 100 =7,000 lb.
total load =11,330 lb. = 5 ton
moment M =5 × 20/8 =12·5 ft. ton.
steel area As =12·5/8 × 1·0 =1·55 sq. in.
total shear S =11,330/2 =5,660 lb.
unit shear s =5660/12 × 6 =78 lb. per sq. in.
and no shear steel is required.

Reinforcement : At top and bottom 2 bars 1 in. = 1·57 sq. in. and 2 section ⅜ in diam., stirrups at 12 in. centres throughout.

Beam Units B5, section 16 in. × 6 in., span 20 ft. 0 in.
dead load beam and slabs =4,330 lb.
live load =20 ft. × 3·5 × 150 =10,500 lb.
total load =14,830 lb. = 6·6 ton
moment M =6·6 × 20/8 =16·5 ft. ton
steel area As =16·5/8 × 1·0 =2 sq in.
total shear S =14,830/2 =7,400 lb.
unit shear s =7,400/12 × 6 =103 lb per sq. in.
value of concrete rib in shear :—
75 × 12 × 6 =5,400 lbs.
distance of critical section from mid-span :—
5,400 × 10/7,400 =7 ft 3 in.
shear resistance
provided =0·22 × 18,000 × 12/6 =7,920 lb.
Reinforcement : at top and bottom 2 bars
1⅛" =2 sq in.
in shear : 2 sections ⅜ in. diam. stirrups at 6 in. and 12 in. centres.

Beam Units B6, section 16 in × 6 in., span 20 ft. 0 in.
dead load beam and slabs =4,330 lb.
live load =20 ft. × 3·5 × 200 =14,000 lb.
total load =18,330 lb. = 8·2 ton.
moment M =8·2 × 20/8 =20·5 ft. ton,
steel area As =20·5/8 × 1·0 =2·5 sq in.
total shear S =18,330/2 =9,160 lb.
unit shear s =9,160/12 × 6 =127 lb per sq. in.
value of concrete rib in shear —
75 × 12 × 6 =5,400 lb.
distance of critical section from mid-span .—
5,400 × 10/9160 =6 ft.
shear resistance provided : 0·22 × 18,000 × 12/4 =11,880 lb.
Reinforcement : At top and bottom 2 bars 1¼ =2·45 sq. in. in shear ; 2 sections ⅜ in. diam stirrups at 4 in. and 12 in. centres.

Constructional details for 20 ft. spans are shown on Figs. 3 and 4 —(*With acknowledgments to "Surveyor"*)

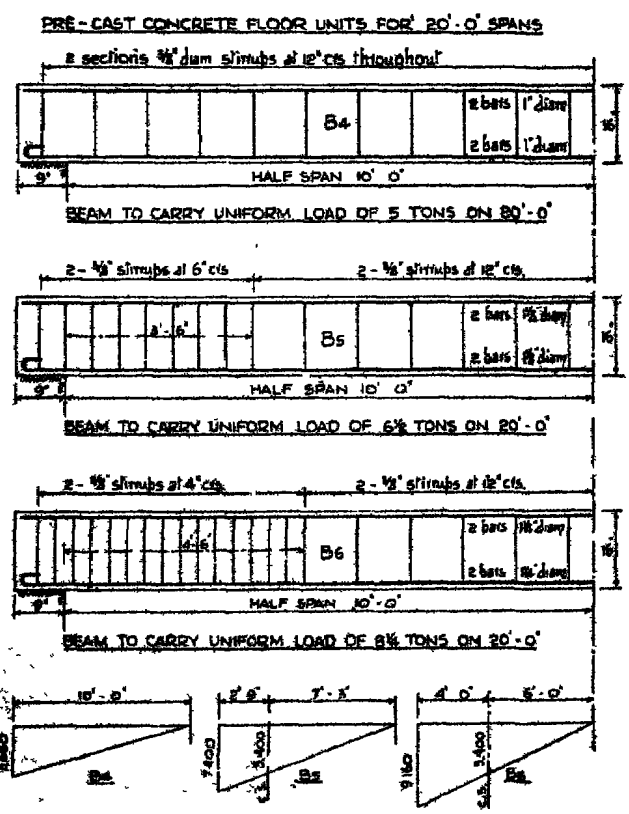
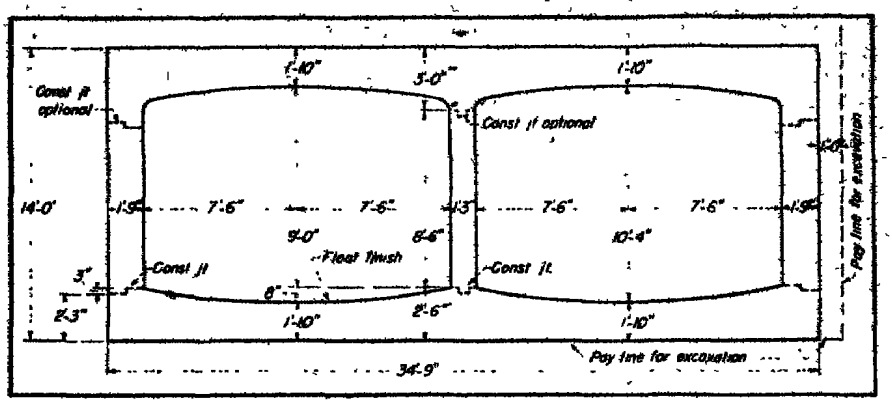


FIG 4



DOUBLE BOX CULVERT is designed unusually heavy to carry airport runways. Contractor pours invert first, then complete superstructure.

CAGING A CREEK WITH TRAVELING FORMS

BOXING Up Coldwater creek in a 4,800-ft double culvert through an extension of Lambert-St. Louis municipal airport is a hit-and-run proposition. Fred Weber, Jr, St. Louis contractor on the job, has learned when to hit and when to run. The box drain follows and fills the present creek channel, leaving no room for by-passing the stream flow Therefore, when the creek is low and docile, Weber hits the job hard with a neat traveling steel form layout But when the creek wants the channel for its exclusive use and goes on a rampage, which it often does, Weber yanks his travelers and everything else loose out of the waterway and runs.

Coldwater Creek drains most of northwest St Louis County. It skirts the south and east sides of the present airport, and meanders right through a proposed extension of the field to the east. Relocation of the stream is impracticable, especially as it already serves as the outlet for all underground drains in the landing field, so it is being caged in a double concrete box culvert that must support long stretches of new runways

Because of the tremendous superimposed loading by big planes, the culvert is unusually heavy in design. Throughout its 4,800 ft. in length it consists of twin boxes, each 15 ft. wide and 10 ft 4 in. high, with a common centre wall. Wall roof and floor thicknesses are shown in the accompanying drawing for the heaviest section under the runways. For sections not under runways the floor, roof and outside walls are thinner by 6 in , though the centre wall remains the same.

Specifications permit the culvert to be poured in three sections invert, walls and roof. The contractor, however, elected to pour in two sections, the invert first,



TRAVELING STEEL FORMS speed up construction of big twin box drain at Lambert-St. Louis airport. Rail-mounted travelers, fitted with jacks, move collapsed form sections ahead.

carried well ahead of the box, and three walls and roof together.

Dragline Excavation

Excavation is comparatively light, consisting mostly of widening and trimming the banks and leveling off the bottom of the old channel. A Lorain crawler crane fitted up as a dragline, easily keeps excavation ahead of concreting. All spoil is piled along the right bank for subsequent backfill, while the top of the opposite bank is kept clear for an access road.

To keep the invert concrete out of the mud, a layer of crushed stone is placed over the creek bed prior to placing of bottom forms and reinforcing.

Blaw-Knox furnished steel forms for both the invert and the boxes. Because inside dimensions of both heavy and light culvert sections are the same, one set of forms is sufficient. Invert forms are simple low sidewall sections.

The twin box forms are independent, except for tying across the top between outside walls, though the two boxes are poured as a unit. Inside forms consist of three 30-ft. sections, outside panels are in 15-ft lengths. Gore sections of various tapers are inserted between form units for curves in alignment.

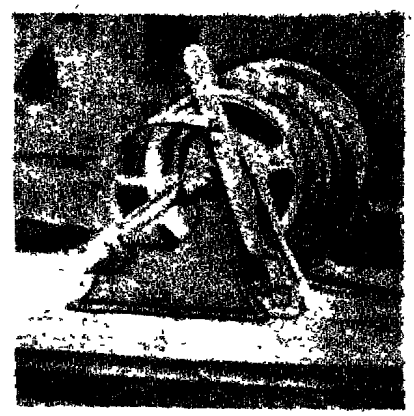
Invert concrete is carried well ahead of the boxes A railroad track is laid on the invert at centre of each barrel to carry 15-ft travelers that move the inner forms ahead. The travelers are fitted with hydraulic jacks for lowering the form sections to clear the side and top. Each section, hinged at top corners, is collapsed by screw jack knee braces The contractor added a hand-operated cable winch to each traveler as moving power.

Outside form panels are handled by a Lorain truck crane, which can lift a 30-ft. section on the near side with a high boom, but only a 15-ft. panel on the far side.

By an ingenious revision of the support system for the outside form panels the contractor, is saving considerable time and money The panels were intended to set on angle brackets fastened to the side of the invert by anchor bolts, which would have been unrecoverable. Besides, the bolts, cast in the invert, would have extended out so far that additional excavation would be necessary at the base of the bank slopes to allow room for moving the invert forms out to clear the bolts in stripping Weber uses the same 6 × 6-in. angle brackets, but has replaced the bolts with lag screws and expansion sleeves. The screws and sleeve assemblies are placed through the invert forms, and the sleeves are incased in the base concrete. In stripping the forms, the lag screws are removed first, which permits a vertical lift on the forms Then the angle brackets for holding the wall forms are attached to the invert concrete by the same lag screws replaced in their respective expansion sleeves. Thus, the only material lost are the sleeves

Readymix Concrete

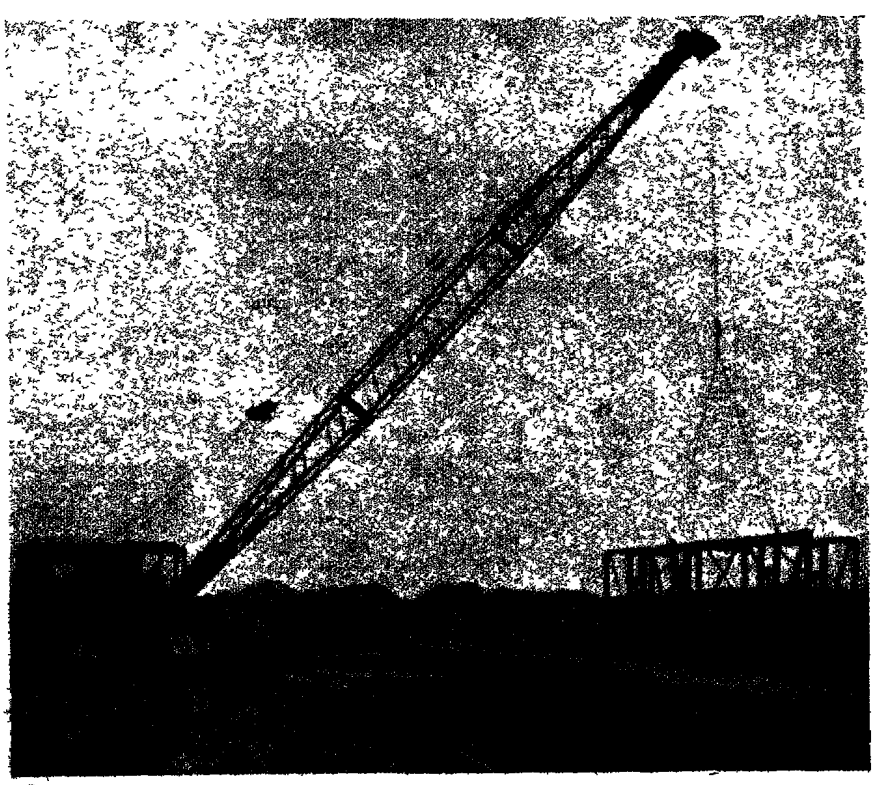
All concrete comes to the job in transit mixers, and is placed by a bottom dump bucket handled by the truck crane The



HAND CABLE WINCH added to travelers eases job of moving forms.

weekly paving schedule on the boxes (90 ft length, 300 cu yd.) is · Pour Monday; move forms and set reinforcing steel Wednesday, pour Thursday, move forms and set steel Saturday Invert sections are poured Tuesday and Friday.

For the city, Milton M Kinsey is president of the Board of Public Service. The job comes under the Sewers and Paving Section, for which Guy Brown is chief engineer. Harold Horan is construction engineer in charge, and J. E. Powers is resident engineer. For the contractor, Ben Taylor is superintendent. —(*With acknowledgments to "Construction Methods"*)



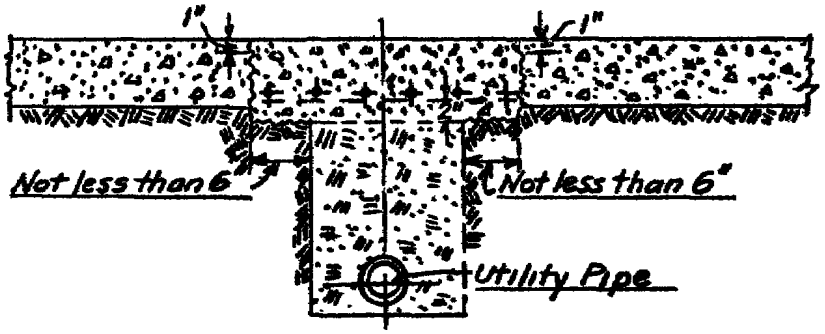
WHEN FLOODS THREATEN, contractor lifts travelers and all other loose items out of cut, for creek uses entire channel on such occasions.

HIGH EARLY STRENGTH CONCRETE FOR UTILITY OPENINGS AND OTHER REPAIRS IN CEMENT CONCRETE PAVEMENTS

Directions are given in detail for repair of openings in concrete surfaces where speed of repair is essential. Materials, preparation of subgrade and of the concrete, and mixing and placing the patch. Data adapted from information supplied

by **F. D. WOODRUFF**

Assistant Superintendent of Maintenance, New Jersey State Highway Department



This illustrates the method of repairing a utility cut.

REPAIR of portland cement concrete pavements on heavily travelled highways requires a working control, not only of the methods to be employed, but of the various aggregates, water content, and admixtures, if used, as well No specific laboratory supervision can be expected Traffic conditions, time element, and sometimes unskilled workmen make it essential that a few simple rules be fixed in order that the very earliest high strength be obtained in the repair of these pavements It is the rule that newly constructed concrete pavements must be cured approximately fourteen days before they are subjected to heavy traffic. It would be extremely disadvantageous to close any modern heavy traffic highway for such a period for the purpose of curing the restoration of a utility opening or repairs to miscellaneous failures

The necessity of maintaining expensive and in many cases dangerous detours, and the disturbance to through traffic, may be almost completely eliminated by the development of high early strength methods The curing time, when proper control is exerted, can be reduced to 18 or 24 hours, and even under adverse conditions to not in excess of 72 hours Where patches are to be opened to traffic in eighteen hours, conditions must be almost ideal, workmanship must be of the best and accelerating admixtures must be employed generally

Surprising results may be obtained by adhering to the following general fundamentals

Cement.—Special cements requiring unusual curing methods and treatment are not readily adaptable to average working conditions Finely ground and specially burned portland cements requiring no treatment other than that given ordinary cement may be used. When these cements are not available, the standard commercial brands of portland cements may be used

Water —The varying acid or alkaline conditions characterizing the average available water, which factors affect the ultimate strength of the resulting concrete, will not be determinable for usual cases in the field Care should be exercised in order that water showing strikingly undesirable characteristics is not used. All other conditions being as nearly perfect as possible, the use of inferior water will as a rule produce poor results

Sand —Sand should be well graded in size, washed clean and completely free from all foreign material Excessively fine material will usually require the use of additional water to produce a workable mix and in all probability will result in inferior concrete

Stone or Gravel —Stone or gravel should contain no loam, vegetable matter or decomposed fragments The size of the coarse aggregate to be employed will depend upon the character and extent of the repair section being treated Small size coarse aggregates are better for small patches, while the standard larger graded aggregates are suitable for larger areas.

Proportions.—Since the determination of the void content of the coarse aggregate may not be made in emergency repairs, an arbitrary mixture should be decided upon for average conditions This should range between 1 : 1½ : 2 and 1 . 1½ · 3½ The richer mix should be used for patches of two square yards or less, and ¾-inch coarse aggregate is preferable for these For patches exceeding three square yards, the larger aggregate and the 1 : 1½ 3½ mix should be used. It is obvious that the smaller the repair section the greater is the effect of impact load insofar as bond may be considered. Where a comparatively large repair area is involved, these impact bond stresses are not exerted so quickly as on the small bond perimeter because of the gradual rolling load, but with the lesser areas the load is immediately transferred to a relatively small area It follows that the smaller repair section should receive more attention and in consequence richer and closer bonding mixtures should be used Whatever mixture may be determined upon, it is essential to increase slightly the cement ratio in the composition of the repair section This increase may be fixed at 10%, although during summer temperatures where admixtures are used, flash set should be closely guarded against. Where the original composition of the pavement being repaired is known, the same mixture may be used in large repair sections

Preparation of the Subgrade

Utility Trenches —Not less than 6 inches of the concrete pavement should be removed beyond the edge of the utility trench The subgrade so exposed, if of the proper stability, should not, in any manner, be adjusted No foreign materials should be added to the subgrade, except where it becomes necessary to remove unstable material for a depth in excess of six inches Where doubt exists as to the quality of the underlying foundations, the normal surface of the subgrade adjacent to the trench should be reduced for a sufficient depth to secure a substantial support

The backfill placed in the excavated trench, of course, must be thoroughly compacted, either by flooding or mechanical tamping Wherever possible, this flooding or tamping treatment of the trench backfill should be carried out in layers not to exceed four inches

Surface Failures —Surface failures may result from the use of inferior materials, poor workmanship, improper subgrade compaction or bad drainage. The repair section should be of such quality as to correct the inferior workmanship and material factors, while a



Quick repair is essential on heavily travelled routes.

simple inspection should determine the necessity of auxiliary lateral drains to overcome any objectionable water levels Where failures have occurred because of insufficiently compressed subgrade, it is essential to remove all material underlying the pavement which may give any indication of instability. Where the removal of the subgrade for a depth exceeding 6 inches becomes necessary, a thoroughly compacted sub-base of broken stone or other suitable material should be substituted To overcome the possibility of such instability extending beyond the limits of the repair section, the subgrade under and adjacent to the section to be patched should be removed as far as possible While this treatment may not be as effective as the removal of additional surface area, it has been found that radiating progressive failures have been almost completely overcome by this procedure.

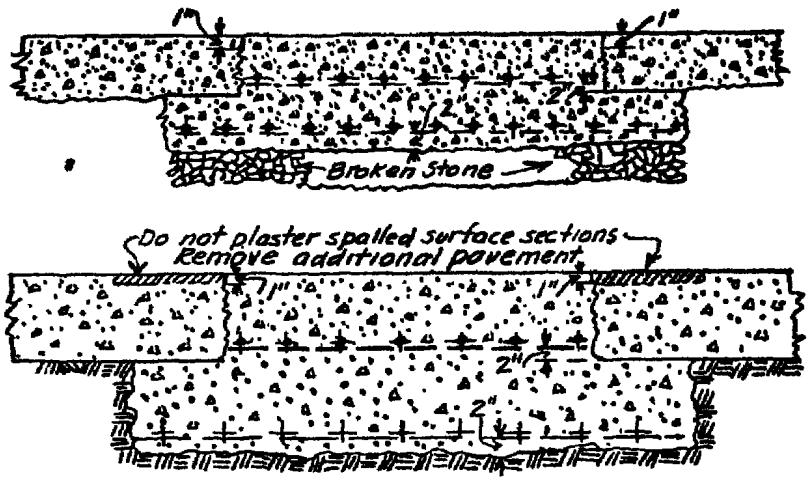
Preparation of Repair Edges—No edges, uniformly plumbed and tooled for the total depth of the pavement, should be permitted where either utility opening or surface failure repairs are to be made The upper section of the adjacent concrete, however, should be plumbed for a depth not to exceed one inch to secure a firm shoulder In some cases, old surfaces have been found where oil film has penetrated to a considerable depth No permanent bond can be expected where this condition prevails, although a rough abutting edge undoubtedly offsets this factor to some extent insofar as bond is involved Where a permanent bond is required, the use of a perfectly plumbed edge invites early failure of the union between the old and new pavement and the consequent "punching out" of the restoration A rough, uneven adjacent edge, properly treated, adds materially to the strength of the repair section and insures more effective bonding Spalled or scaled surface areas of old concrete immediately abutting upon the area to be restored should not be plastered with portions of the mixture placed in the restoration. Such shallow repairs quickly chip out, and thus leave a non-uniform surface and subject the restored area to undesirable impact loads. To obviate this condition it may be necessary to remove sufficient additional old concrete to provide an even surface at the intersection of the old and new concrete

Reinforcement.—Reinforcing steel, preferably $\frac{3}{8}$-inch deformed, should be placed in the restored section 2 inches above the bottom of the slab. The amount of such reinforcement to be used will depend upon the dimensions of the opening being repaired and upon the condition of the subgrade In general the spacing of this reinforcement should conform to the spacing found in the old pavement both transversely and longitudinally. Where reinforced pavements are encountered, either in the repair of utility trenches or failures, the existing reinforcement in the area of the repair should not be removed, but should be bent out of range of repair operations until the section is to be replaced, at which time the steel should be returned as nearly as possible to its original position and tied in with new reinforcement. Where unreinforced pavements are encountered, a mat of bars not more than twelve inches centre to centre should be employed. Such reinforcement is of vital importance in restorations over service trenches.

Method of Placing Concrete Mixture

Treatment of Repair Edges—The edges of the existing pavement, which have been faced vertically for a depth not to exceed one inch measured from the surface, should be thoroughly washed, wire brushed, wetted down and painted with a 1 1 neat cement coating immediately previous to the placing of the concrete mixture. This treatment is quite essential at all times, but obviously more so with high summer temperatures, during which periods the edges must be completely cooled and saturated previous to applying the cement mortar coating, and this should be followed immediately by the placement of the concrete mixture.

Concrete Mixture—Plain—The water content of any plain mixture should not exceed 4 gallons per bag of cement, less



Above, repair when subgrade under sound surface is poor. Below, the damaged surface section cannot be repaired.

the moisture content of both fine and coarse aggregates. During certain periods of the working day, a stock pile being used from the bottom may contain much more water than may be found in the top of the same pile This condition, which is beyond any systematic control, must be carefully watched and the water added at the mixer drum adjusted accordingly. The aggregates, without admixture, should be mixed from $2\frac{1}{2}$ minutes to 5 minutes in a machine mixer No hand mixing methods should be employed. The resultant mixture should flow from a square pointed shovel with difficulty and should show no appreciable slump when dumped upon the subgrade. Any mixture showing more than a one-inch slump in 12 inches should not be used in high early strength repair work. The 4-gallon water-cement ratio mixture will as a rule produce a consistency which is called "unworkable." This "unworkable" mixture should be placed in the repair area in 2-inch layers, each layer being thoroughly compacted by either manual or mechanical manipulation. No initial set should be permitted before the repair in layers is completed This factor is of vital importance and the time of initial set should always be carefully checked. The final "dry" layer of concrete having been placed, it must be tamped sufficiently to bring out a workable surface; the surface should be roughly planed and kept slightly above the finished grade. Initial set will, in average temperatures, occur in from ten to twenty minutes after the final agitation of the mixture, and may be recognized by the sudden recession of the light water film found upon the surface of a well-tamped concrete mixture At the point of initial set the surface should again be tamped thoroughly, planed to grade and finished to the proper surface No proper bond can be expected unless the concrete mixture is reworked at the point of initial set, at which time the total ultimate shrinkage is about to take place Neither can a suitable bond be expected unless the edges of the adjacent concrete are first cleaned, then washed and given mortar treatment

Concrete Mixture—With an Admixture.—Calcium chloride may be used to advantage when necessary to secure high early strength rapidly under emergency conditions Freezing weather and the construction of driveways to hospitals and fire houses and the repair of sections under super-traffic conditions make necessary the reduction of the freezing point and the rapidly increased acceleration of the final set The fact that the use of calcium chloride only slightly reduces the freezing point must be considered, In low temperatures it is quite essential to heat not only the water, but the other aggregates as well Neither should any concrete be placed upon a frozen subgrade

A portland cement concrete mixture containing calcium chloride should contain no more water than that used in plain mixtures, but such mixtures must be manipulated much more quickly than the plain cement combinations Initial set takes place much earlier, especially with high temperatures, and in consequence the depth of the layers to be



Sometimes a truck mixer will speed up repair work

tamped should be increased to not less than four inches The same procedure of finishing as used for plain concrete mixtures should be followed just prior to the initial set, although at low temperatures there appears to be no difference between the action of a calcium chloride treated mixture and an untreated mixture in summer weather The water and fine and coarse aggregates should be heated during freezing temperatures This will insure more rapid hardening of the concrete Where this procedure is followed, temperature conditions must be given utmost consideration and the proportions of admixture, if used, must be carefully controlled The mechanical mixing time should not exceed two and one-half minutes in summer temperatures Where the mixing time is in excess of this period, the mixture may set up in the drum of the mixer, or before the final tamping of the repair section has been completed

Proportions.—For rapid acceleration at temperatures above 70° Fahrenheit, a 1½% calcium chloride solution may be used At temperatures between 50° and 70° Fahrenheit, a 2% solution, 32° to 50° Fahrenheit, a 2½% solution, and at temperatures from 24° to 32° Fahrenheit, a 3% solution No two portland cements will, however, react the same in combination with such an admixture, but for average working conditions these rules can be followed

Preparation of Solution.—The admixture, if calcium chloride, should not be added to the mixer drum in a dry condition. It must be added in solution in order that a thorough distribution of the salts may be assured If added in the dry state, isolated portions containing the admixture may occur and quickly set up, not only fouling the mixer drum, but if placed in the repair section causing areas of unequal initial set To prevent any unequal proportions, the solution should be prepared accurately by some standard method before the admixture is placed in the mixer drum The calcium chloride solution is not to be used in addition to the desired water ratio If a total of 4 gallons of water is required, this must include the calcium chloride solution.

A concrete mixture containing calcium chloride, having been placed and finished to the proper surface, requires no careful curing, but during high temperatures the finished pavement should be protected by burlap saturated at frequent intervals to prevent temperature cracks In plain mixtures the repair sections should be cured with wet hay or by the ponding method until the area is ready to be thrown open to traffic.

It has been found that the colour of the adjacent pavement may be matched in the repair section by the use of carefully adjusted mixtures and by the incorporation of ferrous oxide in the concrete. Such a process is, however, not recommended owing to our lack of knowledge of the reaction of the concrete to such a supplemental admixture in connection with automobile oil drippings and calcium chloride, and owing to the fact that a highly skilled personnel, not as a rule found in general work, must be used to obtain exactness. A high early strength concrete is the ultimate desire, regardless of the fact that a " crazy quilt " surface appearance may be objectionable to the travelling public. Extremely difficult methods of control are necessary to accomplish a blending of colours and it is felt that the early opening of concrete pavements is of far more importance to our vehicular traffic than the problematic results which might be obtained in an effort to please the eye. Where blending of colours is, however, considered necessary, pleasing results have been obtained by determining carefully the composition of the original pavement and incorporating in the patch cement, sand and coarse aggregate from exactly the same sources as applied in the old concrete If the patch is placed as outlined herewithin, there should be no shrinkage of the new concrete away from the old and consequently no reason to paint the perimeter of the repair section with bituminous material This fact, of course, improves the general appearance of the surface —(*With acknowledgments to " Public Works "*)

A FLOATING CONCRETE "ARCH" BRIDGE

A FLOATING BRIDGE, curved in plan, was completed last year across the Derwent river, at Hobart, Tasmania The bridge is 3,165 ft. long, and is composed of 24 reinforced concrete barges, or pontoons, each 132 ft long Each half of the bridge comprises twelve barges between which projecting reinforcement is welded to form two rigid segments At the middle the segments are joined with a pin connection, so that the structure forms a three-hinged horizontal arch. The overall width of the bridge is 40 ft., providing a 30-ft roadway, a footpath 6 ft 6 in wide, and space for a water main

The crown of the horizontal arch is upstream, so that the structure will be in compression, the reaction from the current being taken by the abutments, the barges are normally submerged for a depth of 8 ft 6 in. The only stresses imposed upon the structure are those due to the current and waves, the dead and live loads being carried by the buoyancy of the barges.

The barges, which weigh about 1,000 tons each, were cast on the shore and towed into position in two segments of 12 barges after the projecting reinforcement had been welded and encased in concrete Each barge is divided by reinforced concrete partition walls into 15 compartments. A mooring buoy was anchored at the centre of the site of the bridge, and to this the crown ends of the two segments were temporarily moored, the other ends swung round to the abutments, and pins, 13 in in diameter and weighing ½ ton, were placed in position to fix the ends of the bridge to the abutments A similar pin was used to join the segments at the crown

Variations in the water level range up to 8 ft, and ramps are used between the ends of the bridge and the abutments The floor of the bridge is 4 ft 6 in. above water level

Some damage to the structure was caused by a severe storm after the bridge had been connected, before all the details had been completed, but the damage was not sufficiently severe to prevent the bridge being opened to traffic a few weeks later The size of the waves developed during the storm did not exceed that assumed for design purposes. At times the period of the waves corresponded fairly closely with the natural period of vibration of the bridge in a horizontal plane, and a considerable amount of resonance occurred, particularly in the horizontal oscillations of the bridge The horizontal oscillation was particularly marked at the ends and caused high stresses in the steel girders which anchored the bridge to the abutments The amplitudes due to resonance caused stresses greater than those assumed in the design, and were believed to be the cause of cracking of the concrete in the deck at one end of the bridge. The damaged parts have since been strengthened, and it is stated that a system of prestressing has been installed which increases compressive stresses throughout the structure but reduces tensile stresses. Timber breakwaters are also to be used to reduce the effect of the waves at this end of the bridge. A lift-span is provided to allow the passage of vessels. The design has been patented by Mr. A. W. Knight, Chief Engineer of the Tasmanian Public Works Department —(*With acknowledgments to "Concrete and Constructional Engineering".*)

CANAL LINED WITH STABILIZED EARTH

W. S. BYRNE

Assistant Construction Engineer, U. S. Bureau of Reclamation, Altus, Okla.

CONTENTS IN BRIEF—*To determine practicability of lining irrigation canals with stabilized earth, the U. S. Bureau of Reclamation lined sections of a canal of the Altus project in Oklahoma with soils treated to increase their resistance to erosion and to reduce their permeability Methods used to compact the soils were similar to those used for soil compaction in highway and airport work*



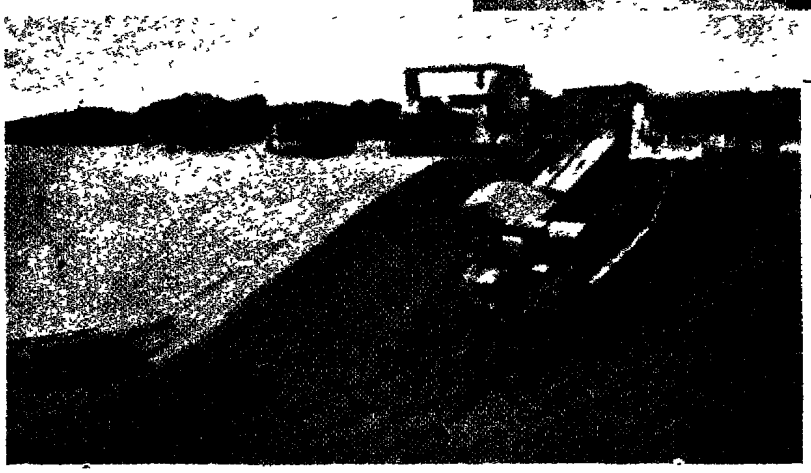


Fig 1. The general procedure in paving the slopes was to spread the stabilized earth mixture with a bulldozer and to compact it with sheepsfoot and pneumatic-tyred rollers.

SECTIONS of the main canal of the Altus Project in Oklahoma have been lined with stabilized earth to determine whether soils can be processed to form a satisfactory canal lining. Portland cement and a resin compound were used as the stabilizing agent, being used separately and in combination in different test sections. Engineers of the Bureau of Reclamation who conducted the tests believe that this is the first time that these two materials have been used in this way

The purpose of the work was to obtain a low cost, relatively impermeable, stabilized earth lining which could be adapted to present day construction machinery and methods This type of lining, if found successful, would be especially adaptable as well as economical in sandy or sandy loam soils where seepage losses would be high and costly, and possibly detrimental to nearby lands, but where the overall cost and the necessity for impermeability would not justify the expense of concrete lining. It was with this theory in mind that the experimental work was carried out.

Various linings tried

A 500-ft. length of the canal was selected in which the soil was stabilized by using cement or the resin compound in varying percentages, and by using a combination of the two. The soil mixtures were placed in 4 and 6-in layers The cement content used in the standard soil-cement mixture was 11 per cent by volume, but was increased to 16 per cent for the small section of plastic soil-cement mix. The resin was mixed with earth in percentages of 1½ and 2½ and placed in the same manner as the soil-cement mixture A mixture of 1½ per cent of resin and 6 per cent cement was tried on a 25-ft section of the canal slope In all cases the soil used consisted of the sandy loam excavated from the canal section

The resin compound used in these tests was Stabinol, which was placed on the market in 1944 by the Hercules Powder Co (*ENR* March 16, 1944, vol p 385).

The method of constructing the lining is illustrated by the accompanying photographs After the soil and cement (or resin) had been mixed in a 2-cu yd mixer near the job it was dumped at the top of the slope by an end-dump truck Spreading of the material was by a bulldozer

A sheepsfoot roller, pulled up and down the slope by means of a winch on the rear end of a tractor, was used in the first operation to compact the lining Final rolling was by means of a pneumatic-tyred unit pulled by still another tractor using a snatch block attached to a dead end

An exception to the methods just described was construction of the short section of plastic soil-cement lining

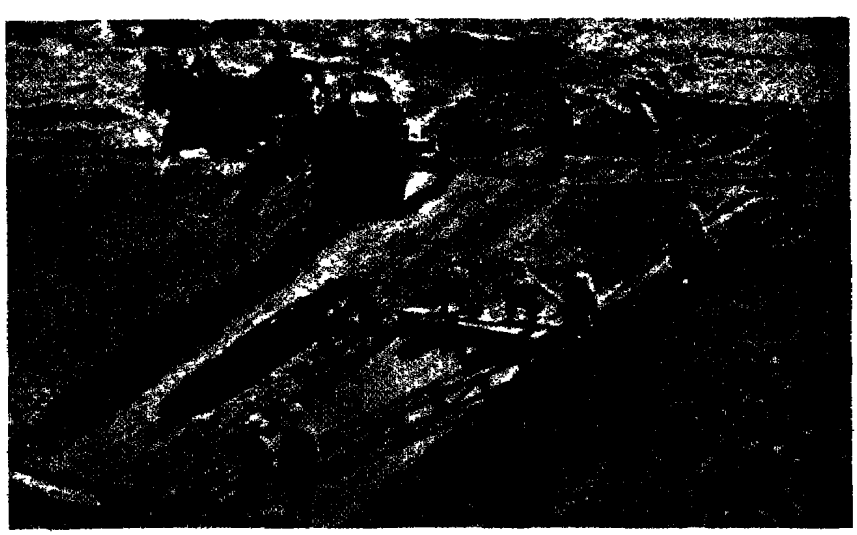


Fig. 2. A plastic soil-cement mix at about 6-in. slump was placed on one of the test sections in the same manner as concrete, using forms and hand tools to produce a lining of uniform thickness. No compaction is required with this type lining

The soil, cement, and water were mixed in the same 2-cu. yd. mixer at about a 6-in slump instead of at optimum moisture content, as is the case with standard soil-cement mix, and delivered to the slope of the canal in a concrete bucket handled by a crane (Fig. 2) Workmen using a screed and hand floats spread the material, which requires no additional compaction

The completed lining on the slope and the successive stages of placing the lining in the canal bottom are shown by Fig. 3 The method used in lining the canal bottom was similar to normal stabilizing methods on highway and airport work To avoid confusion in studying Fig 3, it should be noted that the completed soil-cement lining has been covered with a 1-in earth blanket for curing, while the resin mix has no covering.

Need for special equipment indicated

Since the amount of lining was so small and strictly on an experimental basis, representative cost data are not available. The experiment was carried out with equipment on hand on the project, and the prime purpose of the experiment being to construct a lining of the best possible quality rather than attempting to develop methods or special equipment at this time. But it has been concluded that if this type of lining proves successful from the standpoint of durability, special equipment will need to be developed to eliminate the awkwardness of working on the slope of the canal bank. Such a development will be necessary to place the cost of installing this type of lining on a competitive basis with more costly linings, such as concrete.

The canal has been operating with a water depth of approximately 6 ft since Sept 11, 1945, and to date no serious damage has been noted on any section of the lining However, it will be necessary to observe the reaction of the test lining through several operating seasons before any prediction can be made as to its durability as compared with other types of canal linings.

All construction work on the Altus Project is being carried on by the Bureau of Reclamation forces under the general supervision of Michael W Straus, commissioner, and Walker R. Young, chief engineer. Wesley R Nelson is regional director with headquarters in Amarillo, Texas, and H. E Robbins is construction engineer with headquarters on the project.

The experimental construction was directed by the writer and K. L Powers, materials engineer. The success of the work was greatly enhanced by the help of V. O. Peterson of the soil-cement bureau of the Portland Cement Association, P. J. Reno of the Hercules Powder Co, and W. G Holtz of the Bureau of Reclamation's earth materials laboratory in Denver. M. D Catton, manager of the soil-cement bureau, and R. F. Blanks, chief, engineering and geological control and research division of the Bureau of Reclamation, also contributed valuable suggestions —(*With acknowledgments to "Engineering News-Record."*)



Fig. 3. In lining the bottom of the canal the stabilizing materials were delivered in bags and spread on the surface with hand tools as shown at the top. Mixing and compaction was by methods common to soil-cement paving operations on airport and highway work.

PRESTRESSED CONCRETE SLEEPERS

A NEW method of manufacturing precast prestressed reinforced concrete units is employed at the works of Dow-Mac Products, Ltd , at Tallington, near Stamford, Lincs The factory, which is over 500 ft. long and 140 ft wide, is at present planned for the mass production of railway sleepers and



Fig. 1.—View of Conveyors and Reinforcement Stretching Machine



Fig 2.—Assembling and Filling Moulds

has a potential daily output of 300 sleepers All concreting operations take place at the centre of the factory, and from this position longitudinal conveyors extend in each direction for 225 ft There are ten parallel lines of longitudinal conveyors each of which take a double line of sleepers , each conveyor therefore accommodates 100 main line sleepers At each end of each longitudinal conveyor there is a pair of anchor plates for securing the wire reinforcement. *Fig.* 1 is a general view of the factory taken from the prestressing end of the conveyors

The procedure is first to stretch the wire reinforcement by the method subsequently described The wires extend the full length of the conveyors Adjacent to the central concreting plant, the steel moulds, which have been cleaned and coated with an emulsion, are brought on to the conveyors The prestressed reinforcement passes through the moulds, which are fitted with slotted end-plates. Two lines of sleepers are dealt with simultaneously An empty mould is pushed on to a vibrating table immediately below the hopper containing the concrete and while it is being vibrated the mould is filled as shown in *Fig.* 2 The filled mould is now passed on to the continuation of the longitudinal conveyor along which it travels to the far end, or to within 4 in of the preceding filled mould , in so doing it slides along the prestressed wires In this position (*Fig* 3) the exposed face of the sleeper is trowelled and the concrete is again vibrated

The sleeper is now left to harden The end-plates are removed from the moulds and when the sleepers have hardened sufficiently the wires between adjacent sleepers are burned off close to the face of the concrete. The sleepers then travel along the conveyors towards the centre of the factory again where they are mechanically lowered off the longitudinal conveyor on to a transverse conveyor at a lower level which carries them clear of the lines of longitudinal conveyors. At the end of the transverse conveyors the sleepers is lifted from the mould (*Fig.* 4) and proceeds directly to another conveyor which transports the unit to storage outside the factory. The first sleeper from each line is tested in a manner complying with British Standard 986 (1945) ; the testing operation is illustrated in progress in *Fig.* 4.

One feature of this factory is the small number of people employed All movements of the moulds, sleepers, and materials within the factory are mechanically controlled The aggregates are obtained from the Company's pits adjacent to the factory, and are delivered washed and graded Elevators discharge the aggregates into separate hoppers for $\frac{1}{2}$ in , $\frac{1}{4}$ in , and sand Rapid-hardening normal-setting Portland cement is delivered in bulk and is brought to the batch-weighing plant by a screw conveyor The concrete has a water-cement ratio of 0·39 and is mixed in pan-type mixers, which discharge into a travelling skip by which the hopper, from which the moulds are filled, is replenished The hopper and the vibrating table travel transversely across the lines of the longitudinal conveyors to a position in the line of sleepers to be concreted

Prestressing Operations.

In each sleeper there are 18 or 20 No 5 gauge steel wires, arranged in three layers The wires have a minimum ultimate strength of 100 tons per square inch and a proof stress of 72 tons per square inch The coils of wire are placed on the spools shown in *Fig* 1 The free ends are passed through the steel anchor plates and are pulled by a winch to the remote end of the conveyor line, where they are fixed by malleable iron wedges in anchor-plates similar to those shown in *Fig* 1 Since the effectiveness of the reinforcement depends upon the bond between the concrete and the steel, measures are taken to



Fig. 3.—Surface Vibration and Trowelling.

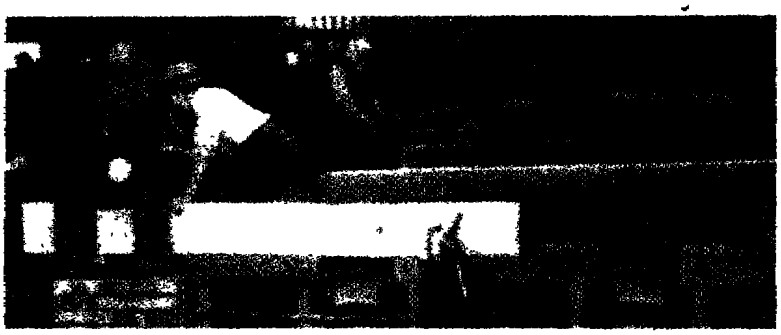


Fig 4—Testing Sleepers

ensure that the wires are clean and free from rust by passing them through a vibrated box, seen in *Fig.* 1, containing carborundum granules The wires are then secured by wedging in the near-end anchor plates The wires projecting from the latter are severed from the coils and are then prestressed in pairs To the end of each wire is attached a self-tightening clamp (*Fig* 5), which is fixed to the ram of a long-travel pneumatic jack which has hydraulic cushioning The pulling force is applied gradually until it reaches 2·2 tons on each wire (corresponding to a stress of about 65 tons per square inch), at which load the power is automatically cut off The extension of each wire is about 2 ft 3 in on a length of 450 ft. The wedges in the near-end anchor-plates, which become loose under the stretching process, are now driven in tight and a white paint mark is made on each wire at the extremity of the wedges The stretching force on the wire outside the wedges is now relaxed The paint mark would indicate if any slip had occurred, but the wedging is sufficient anchorage to retain the wires at the full amount of the predetermined prestress.



Fig. 5.—Stretching a Pair of wires.

The effectiveness of this method of manufacture has been proved not only by the routine tests, but by the fact that several thousand sleepers made by this process have been used in service. In a paper entitled "The Development and Manufacture of Prestressed Concrete Units," read before the Society of Engineers last month by Mr R S. V. Barber and Mr. D. R Lester, it was stated that prestressed sleepers, similar to those described here, were laid in 1942 in the main line of the L M.S R at Chedington where speeds of ninety miles an hour are attained. Many sleepers of this type have been laid in main line at Hitchin in 1943 for trial by the L N E R., and others have been supplied to the G.W.R and to the Southern Railway as well as to railways in India, Egypt, New Zealand and Holland.

Examination shows that the exposure of the steel wires at the ends of the sleepers is not detrimental. The behaviour of the sleepers in service and under test before and after service appears to be highly satisfactory The initial vibration can be considered merely as an aid to filling the mould with the correct amount of compacted concrete of very dry mix, while the second vibrating operation, which takes place within a minute or two of filling the mould, is the real consolidation process preceding the maturing of the sleeper during which stage the unit is undisturbed —(*With acknowledgments to "Concrete and Constructional Engineering".*)



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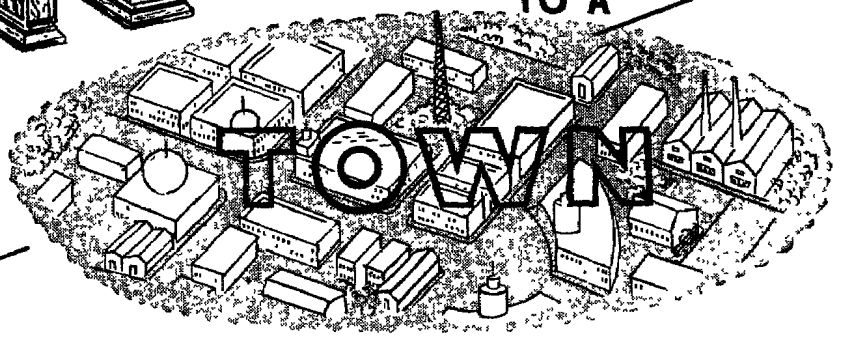
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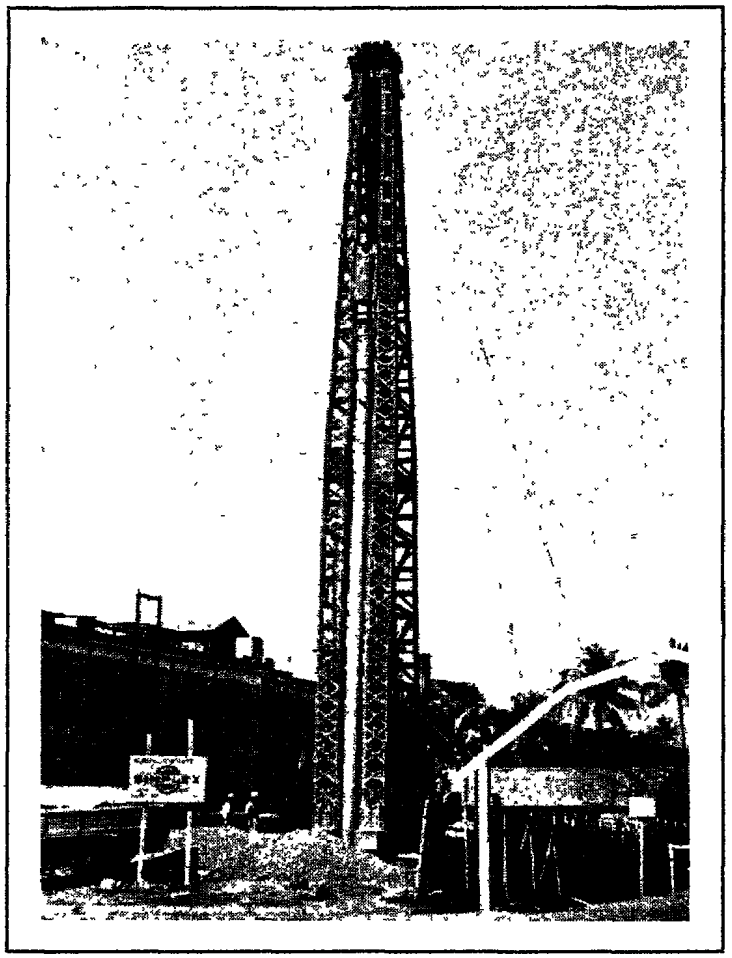
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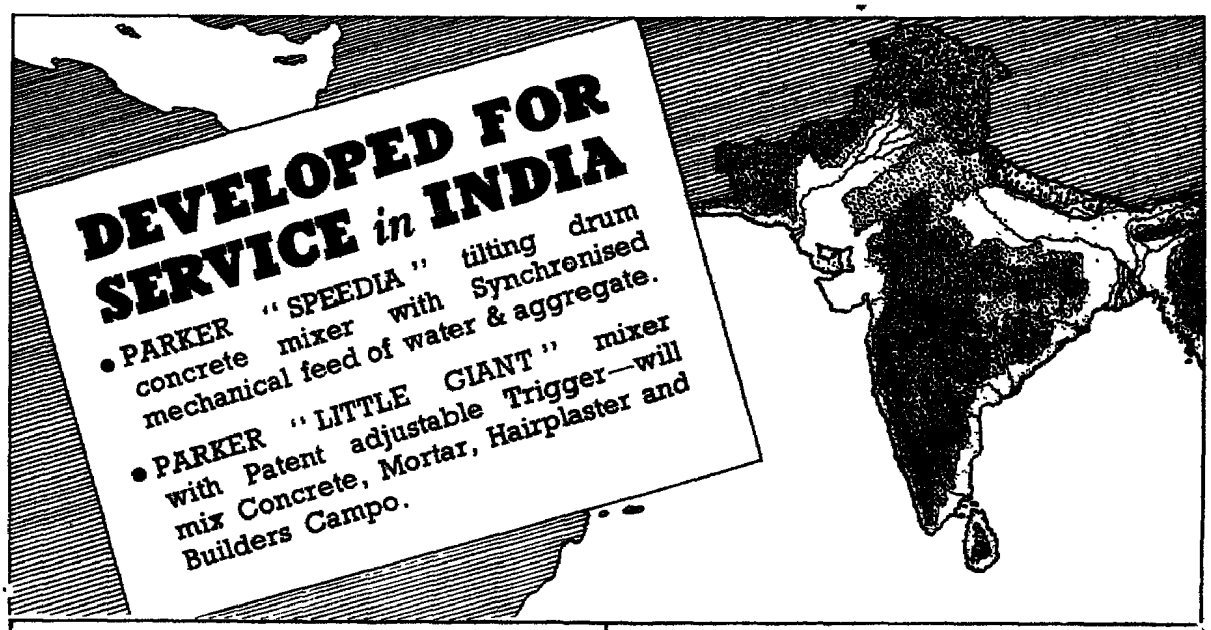
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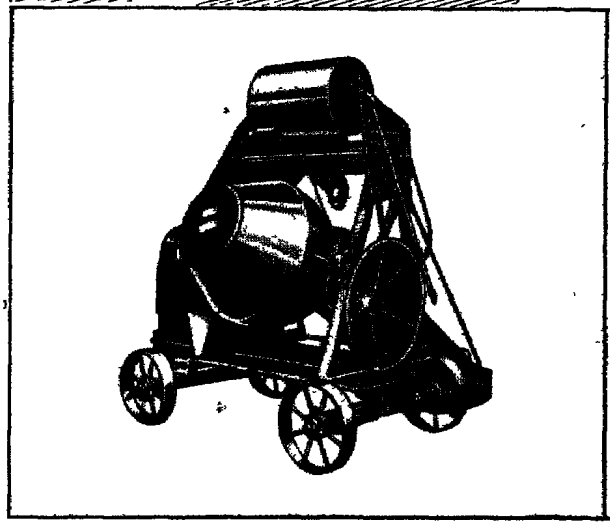
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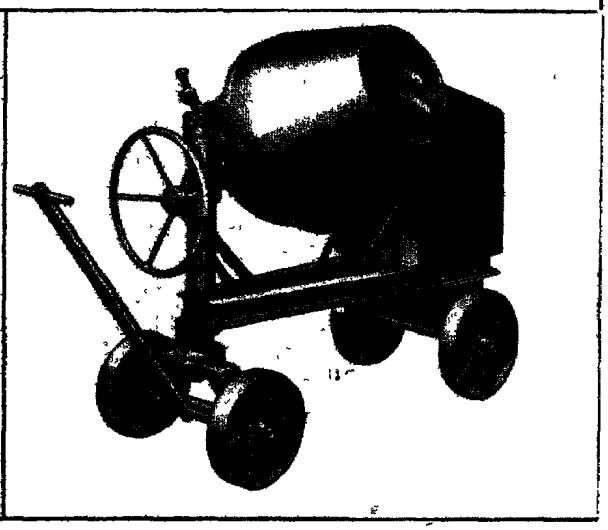
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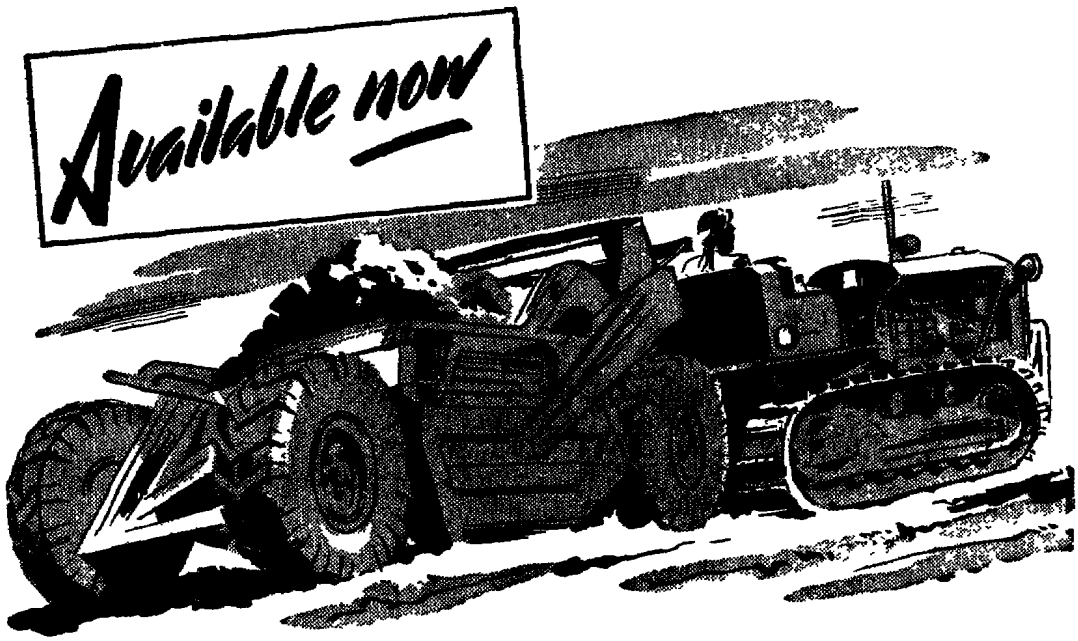
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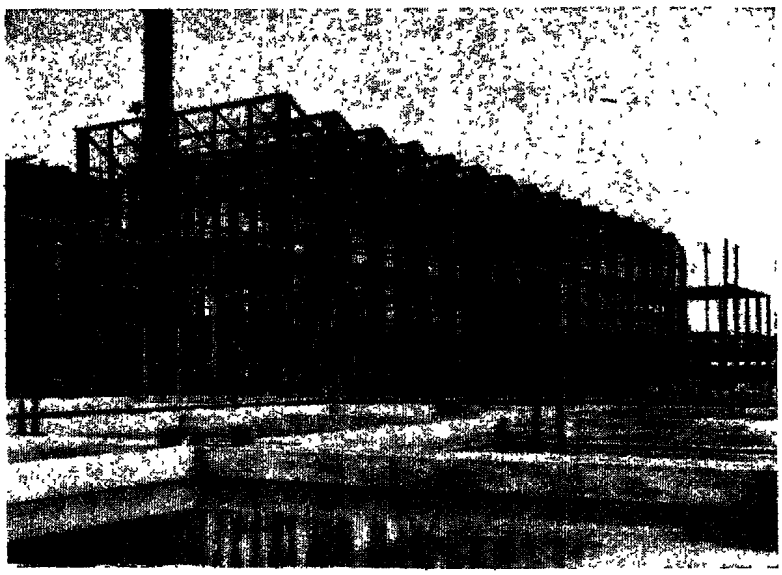
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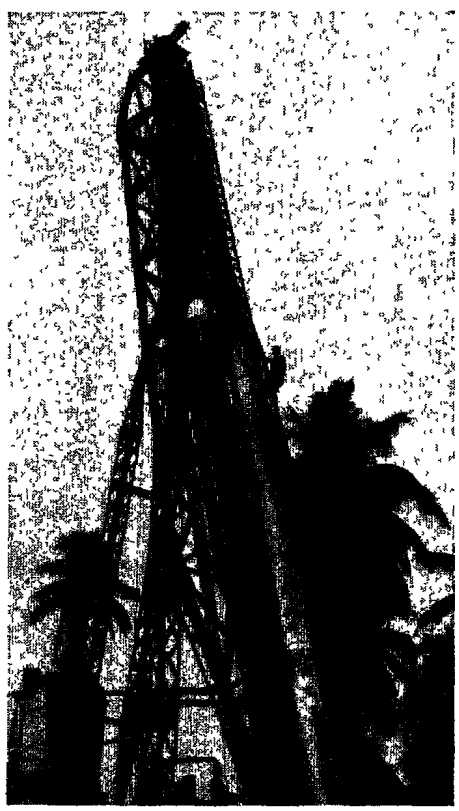


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JAHANIA
JALJON DOABA
JAMMU
JALALABAD
JALLO
JAITU
JANDIALA
JANIWALA
JARANWALA
JHANG MAGHIANA
JHELUM
JOGINDERNAGAR
JULLUNDER CITY & CANTT.
JUTOGH
JWALAMUKHI ROAD
KHEWRA
KABUL
R S. Channan & Peshawar.
KABUL RIVER
KAHALIA
KALAKE
KALKA
KALABAGH
KALLUR KOT
KAMOKE
KANGANPUR
KANGRA
KAPURTHALA
KARTARPUR
KARUR
KASAULI
R. S Kalka
KHAIRABAD
KHUSHAB
KASUR
KHANEWAL
KHAROR PAKKA
KOT RADHAKISHEN
KANDAGHAT
KUNDIAN
KHANNA
KHANPUR
KHASA
KOT ADU
KHARIAN
KILA RAIPUR
KOHAT
KOT KAPURA
KOT LAKHPAT
KURALI
LAHORE & LAHORE CANTT.
LAKHEWALI
LAKHMARWAT,
LALAMUSA
LALIAN
LODHRAN
LOHIAN KHAS
LEIAH
LUDHIANA
LYALLPUR
MACLEOD GUNJ ROAD
MAGHARWAL
MALISI
MAJITHA
MALAKWAL
MALERKOTLA
MAKHU
MALOUT
MALSIAN SHAKOT
MANDI BAHAUDDIN
MANDRA
MANSA
MARH BALOCHAN
MANDI BUREWALA
MANDI STATE
MANGWAL
MARDAN
MAMU KANJAN
MAUR
MAINWALI
MIRPUR
R. S. Sarai Alamgir
MIRAN SAHIB
MOGA
MOGHALPURA
MONTGOMERY
MORINDA
MUKERIAN
MUKTSAR (B.B.&C.I)
MULLANPUR
MULTAN CITY & CANTT.
MUREE
R. S. Rawalpindi.
MUZZAFFARGARH
MURIDKE
MIANCHANNU
MINCHINABAD
MIRAN SAHIB
NAKODAR
NANKANA SAHIB
NARANG
NAROWAL
NAWANSHAHR DOABA
NOWSHERA
NURKOT
NURMAHAL
NURPORE ROAD
NAGROTA
OKARA
PALAMPUR
PARACHINAR (*Via Thal*)
PASRUR
PATHANKOT
PAKPATTAN
PATTI
PINDIGHEB
PINJOUR
PATTOKI
PESHAWAR
PHAGWARA
PHILLAUR
PHULERWAN
PIRMAHAL
PIND DADAN KHA
PIPLAN
QAIDAN
QILLA SHEIKHUPURA
QILA SOBHA SINGH
RAHON
RANBIRSINGHPURA
RAFKOT (*R. S Mullanpur*)
RAHWALI
RAIWIND
RAHIMYARKHAN
RAJA JANG
RANBIR SINGHPURA
RORANWALA
RAWALPINDI
RENALA KHURD
RUPAR
RURALA ROAD
SADIKABAD
SAMBRIAL
SAILA KHURD
SANAHWAL
SANGLA HILL
SILLANWALI
SARA ALAMGIR
SARGODHA
SARNA
SATNAUR BADESRON
SHAH ALAM
SHAHDARA
SHARAKPUR
(*R. S Shahdara*)
SHAHPUR SADAR
SHAH CHAURASI
SHORKOT ROAD
SHUJABAD
SIALKOT
SIHALA
SIMLA
SIRHIND
SOLON
SUKHO
SULTANPUR LODHI
SUKHEKE
SRINAGAR
R. S. Hasilian or Rawalpindi.
TALAGANG (*R. S. Chakwal*)
TALWANDI
TARN TARAN
TANDA UDMAR
TANDIANWALA
TANK
TAXILA
THERI SANSI
TEHSIL SHAKARGARH
THABALKE
THAL
TOBA TEK SINGH
VEHARI
VEKKA
WAZIRABAD
WARBURTON
WASAWEWALA

NEW DELHI AREA :—

ACHNERA
AGRA
ALAL
ALIGARH
AMROHA
AMBALA CITY
AMBALA CANTT
ATRAULI ROAD
BABRALA
BARHAN
BARARA
BARNALA
BAGHPAT ROAD
BAH (*R. S. Bajangaj*)
BAHADURGARH
BAHJOI
BARETA
BASIKIRATPUR
BASIKRATPUR
BEGAMABAD
BHIWANI
BHUCHCHU
BALLABGARH
BARAUT
BUDHLADA
BULANDSHAHR
BIJNOR
BRINDABAN
CHANDAUSI
CHANDPUR SIAU
CHHINTANWALA
CHOLA
DADRI
DALMIA-DADRI
DIBAI
DEHRA DUN
DELHI
DELHI SHAHDRA
DHOBAND
DANKAUR
DAURALA
DHAMPUR
DHANARI
DOIWALA
ETMADPUR
FARIDABAD
FEROZABAD
GANAUR
GANGOH
GARHMUKTESAR
GHAZIABAD
GHARAUNDA
GOT
GOHANA
GURGAON
GULAOTHI
HALDAUR
HANSI
HARDWAR
HAPUR
HATHRAS
HISSAR
HASANPUR (*R S Gajraula*)
JAGADHRI
JAKHAL
JALESAR ROAD
JAWALAPUR
JIND CITY
JULANA
KALANWALI
KANTH
KANDHLA
KAURARA
KHATAULI
KNEKRA
KHURJA
KARNAL
KAITHAL
KASHIPUR
KASIMPUR KHERI
KOSI KALAN
KOSLI
KOTDWARA
KURUKSHETRA
LEHRAGAGA
LHAKSAR
LOHARU
MANDI DHANAURA
MADLAUDA
MANGLORE
MANSURPUR
MEERUT
MILAK
MORADABAD
MOWANA MANDI O/A
MUNDLANA
MURADNAGAR
MURSAN
MUSTAFABAD
MUITRA
MUZAFFARNAGAR
NAJIBABAD
NAGARIA SADAT
NAGINA
NABHA
NANAUTA
NARELA
NARWANA
PALWAL
PANIPAT
PATIALA
PILKHUWA
RAIWALA
RAJA-KA-SAHASPUR
RAJGHAT-NARORA
RAJPURA
RAMAN
RAMNAGAR
RAMPUR
RAMPUR MANHYA RAN
RAMPURA PHUL
RAYA
REWARI
ROHANA KALAN
ROHTAK
ROORKEE
RIKHIKESH
SAFIDON
SAHARANPUR
SAMPLA
SANGRUR
SASNI
SUNAM
SAMALKHA
SAMBHAL HATIM SARAI
SARSAWA
SEOHARA
SHIKOHABAD
SHAHABAD MARKANDA
SHAMLI
SIRSA
SONEPAT
SIMBHAOLI
TAPA
TAPRI
TARAWARI
THANA BHAWAN
THANESAR
TUNDLA
TOHANA
UCHANA
UKLANA

CAWNPORE AREA :—

ACHALGANJ
ACHHALDA
AGSAULI
AHIMANPUR
AHRAURA ROAD
AJGAIN
AJODHYA
AKBARPUR
ALIGANJ
ALLAHABAD
AMETHI
AMOUSI
ANJHI
ANUPGANJ
AONLA
ASAFFUR
ATARRA
AURAIYA
AUNRIHAR
AZAMGARH
BABHNAN
BACHRAWAN
BADLAPUR
BADSHAHPUR
BAHERI
BAHRAICH
BAHRAMGHAT
BAIBWARA
BAITALPUR
BAKSHA
BALAMAU
BALLIA
BALRAMPUR
BAMIANA
BANBASSA
BANDA
BANGAR MAU
BARA BANKI
BAREILLY
BARHAJ BAZAR
BARHNI
BARHALGANJ
BASAI
BASHARATGANJ
BASTI
BELLARMIN
BELTHARA ROAD
BENARES CANTT.
BENARES CITY
BEWAR
BHADOHI
BHARTHANA
BHATPARRAN
BHATNI
BHITAURA
BHITI
BHONGAON
BIJAURA
BILGRAM
BILPUR
BAINDKI ROAD
BISALPUR
BISWAN
BODARWAR
BRIDGMANGANJ
BUDAUN
BURHWAL
CAMPIERGANJ
CAPTAINGANJ
CAWNPORE
CHANDAULI MANJHWAR
CHANDWAK
CHAURICHURA
CHEITAUNI
CHHIBRAMAU
CHILBILA
CHILH
CHILWARIA
CHIRGAON
CHIT-BARAGAON
CHOWKGHAT
CHUNAR
COLONELGANJ
DARYABAD
DARYAOGANJ
DATIA
DHEENA
DIBIAPUR
DILDAR NAGAR
DOBHI
DOHRIGHAT
DULAHPUR
ETAH
ETAWAH
FARRUKHABAD
FATEHGARH
FATEHPUR
FYZABAD
GAHMAR
GANJDUNDWARA
GAURIGANJ
GHAZIPUR
GHOSI
GHUGHLI
GOLA GOKARAN NATH
GONDA
GORAKHPUR
GOSHAINGANJ
GURSHAHAIGANJ
HAIDARGARH
HALDWANI
HANDIA KHAS
HARDOI
HARGAON
HARNAMGANJ
HARPALGANJ
HARPALPUR

Cawnpore Area (Contd.)

JAHANAGANJ ROAD
JAIS
JAKHANIA
JALALPUR
JANGHAI JN.
JANGIGANJ
JARWAL ROAD
JASWANTNAGAR
JAUNPUR
JHONATHPUR
JHANSI
JHINJHAK
JHUSI
KARRAI
KACHHLA GHAT
KACHHWA ROAD
KADIPUR
KAIMGANJ
KALPI
KAMALGANJ
KAMALPUR
KANAUJ
KANCHAUSI
KARARI
KARENGI
KARWI
KASGANJ
KASHI
KATHGODAM
KATHKUIYAN
KATKA
KAWAPUR
KHORIPUR
KERAKAT
KHADA
KHAGA
KHALILABAD
KHATEMA
KHETA SARAI
KHORASAN ROAD
KICHHA
KIDIHDAPUR
KOHNDAUR
KONRH ROAD
KOPAGANJ
KUNCH
KUNRAGHAT
KUSMHI
LACHHMIPUR
LAKHIMPUR KHERI
LAKSHMIGANJ
LALGANJ
LALITPUR
LAR ROAD
LUCKNOW
MADHOGANJ
MADHOSINGH
MAHMUDABAD
MAHOBA
MAHOLI
MAHPUR
MAINPURI
MAKRANDPUR
MALIHABAD
MALIPUR
MALLAWAN
MANAURI
MANDAH ROAD
MANKAPUR
MANPUR NAGARIA
MARHRA
MARIAHU
MASKANWA
MAU AIMA
MAU NATH BHANJAN
MAURANIPUR
MEJA ROAD
MIRANPUR KATRA
MIRZAPUR
MISRIKH TIRATH
MOGHAL SARAI
MOTH
MUHAMMADABAD GOHNA
MUNDERWA
MUSAFIRKHANA
NAGARIA SADAT
NAIPALGUNJ ROAD
NANDGUNJ
NANPARA
NAUTANWA
NAWABGUNJ GONDA
NIBKAROR
NOWGARH
ORAI
ORCHHA
PACHPERWA
PADRAUNA
PARIAWAN
PARTABGARH
PATIALI-ON-GANGES
PAYAGPUR
PEPPEGANJ
PHAPHUND
PHARENDA
PHARIHA
PHEPHNA
PHULPUR
PIPRAICH
PITAMBERPUR
PILIBHIT
PUKHRAYAN
PUNCH
PURANPUR
RAGAUL
RAGHURAJ SINGH
RAI BARELI
RAJA-KA-RAMPUR
RAJA TALAB
RAJAWARI
RAMKOLA
RANI-KI-SARAI
RASAULI
RASRA
RASULABAD
REOTI
RICHHA ROAD
RISIA
RUDAIN
RUDAULI
RURA
SAADAT
SAFDAR GANJ
SAFIPUR
SAHATWAR
SAHAWAR TOWN
SAHJANWA
SAIDPUR BHITR
SAIYEDRAJA
SAKALDIHA
SALIMPUR
SANDILA
SARAIMIR
SARDARNAGAR
SARJU
SHAHGANJ
SHAHGARH
SHAHJAHANPUR
SHAMSHABAD
SHEIKHUPUR
SHIVPUR
SHOHRATGARH
SHRI KRISHNA NAGAR
SIDHAULI
SIKANDARPUR
SIKANDRA RAO
SISOTARGHAT
SISWA BAZAR
SITAPUR
SORON
SULTANPUR
SURAIMANPUR
SURIAWAN
TAHSIL DEORIA
TAHSIL FATEHPUR
TAKIA
TAMKOHI ROAD
TANAKPUR
TANDA
TARAON
TARIGHAT
TILHAR
TIKRI
TULSIPUR
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UNAO
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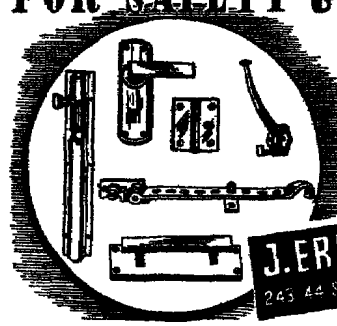
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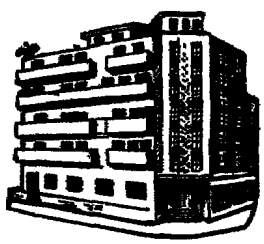
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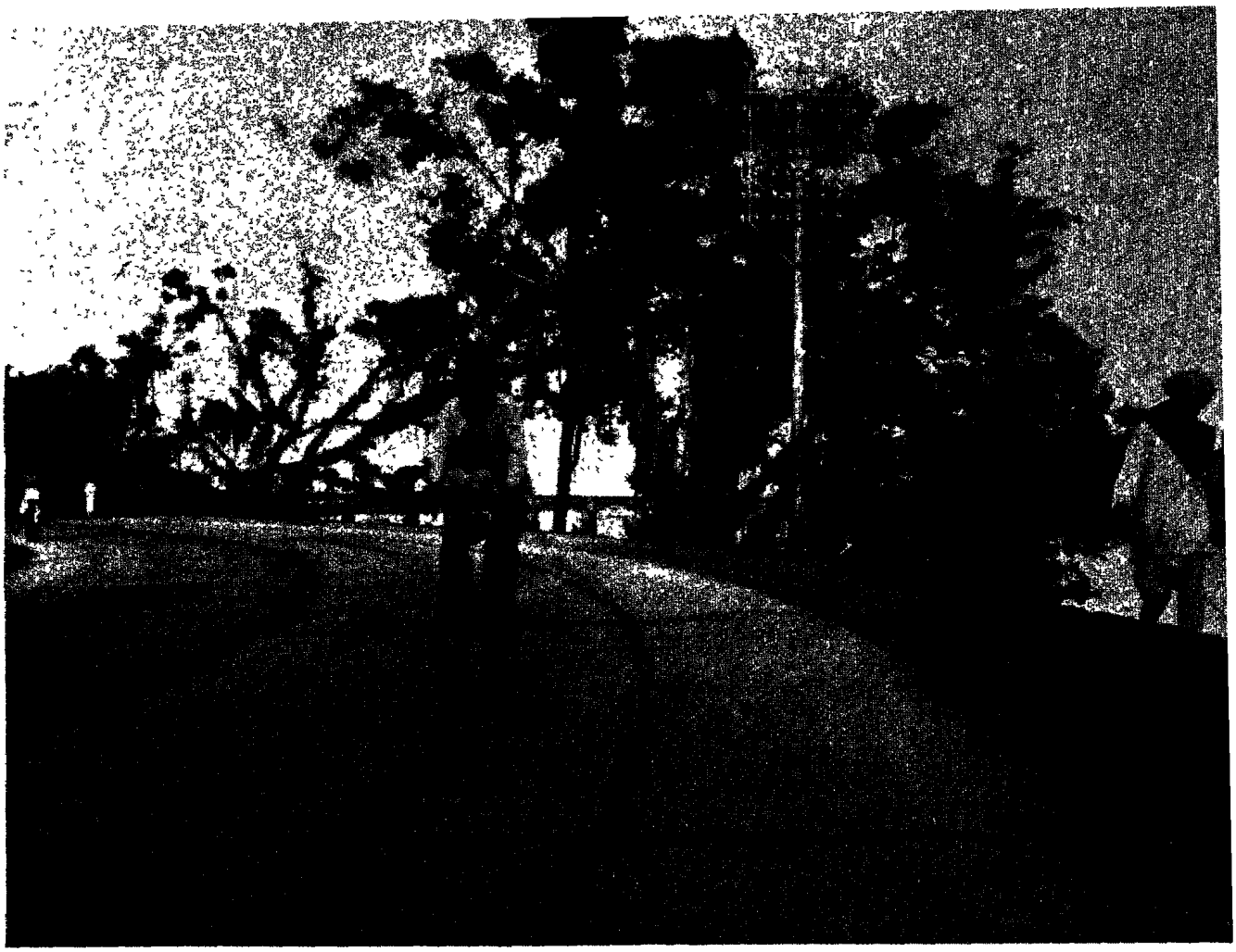
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Width	. . .	30 ft.
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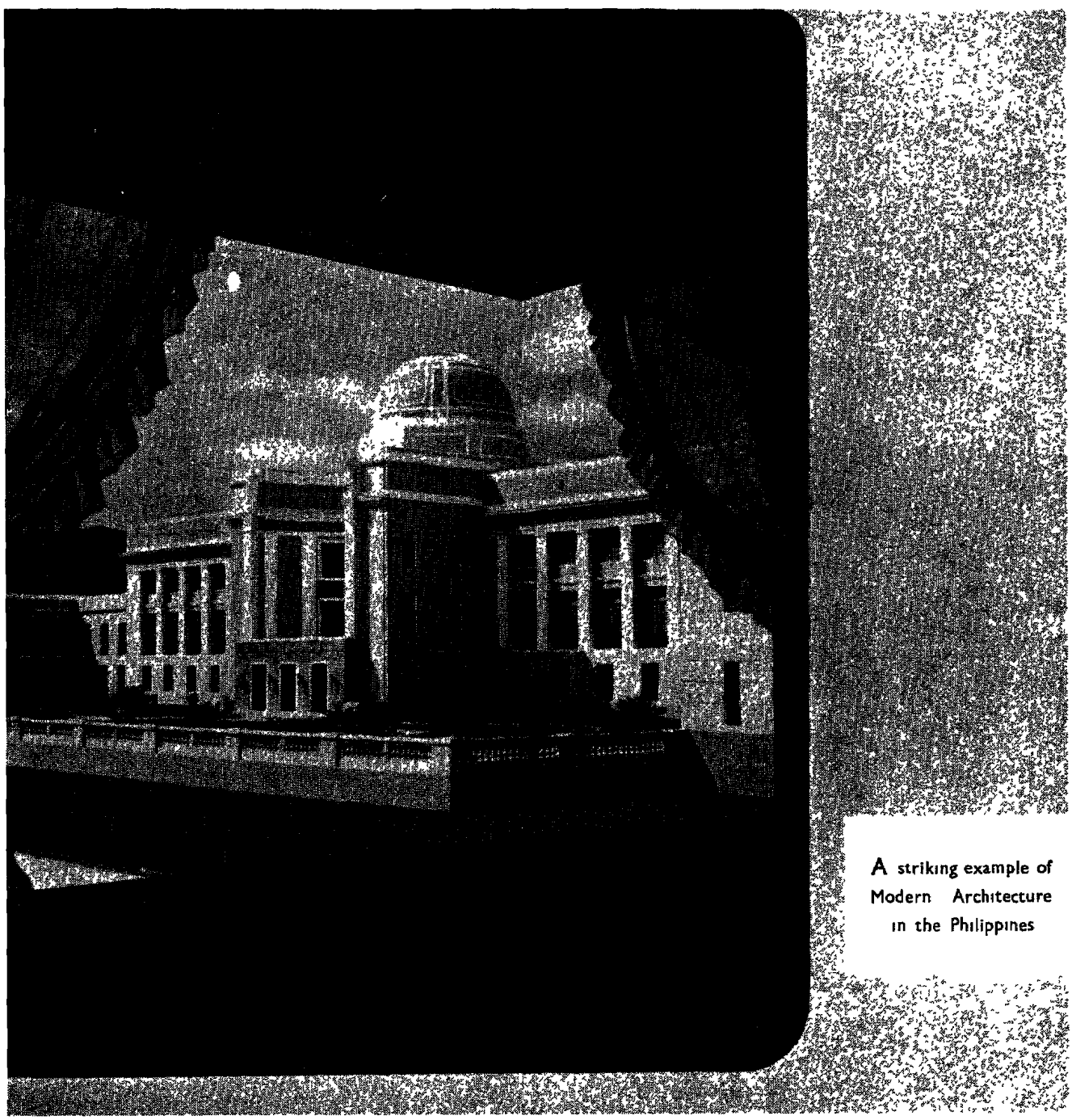
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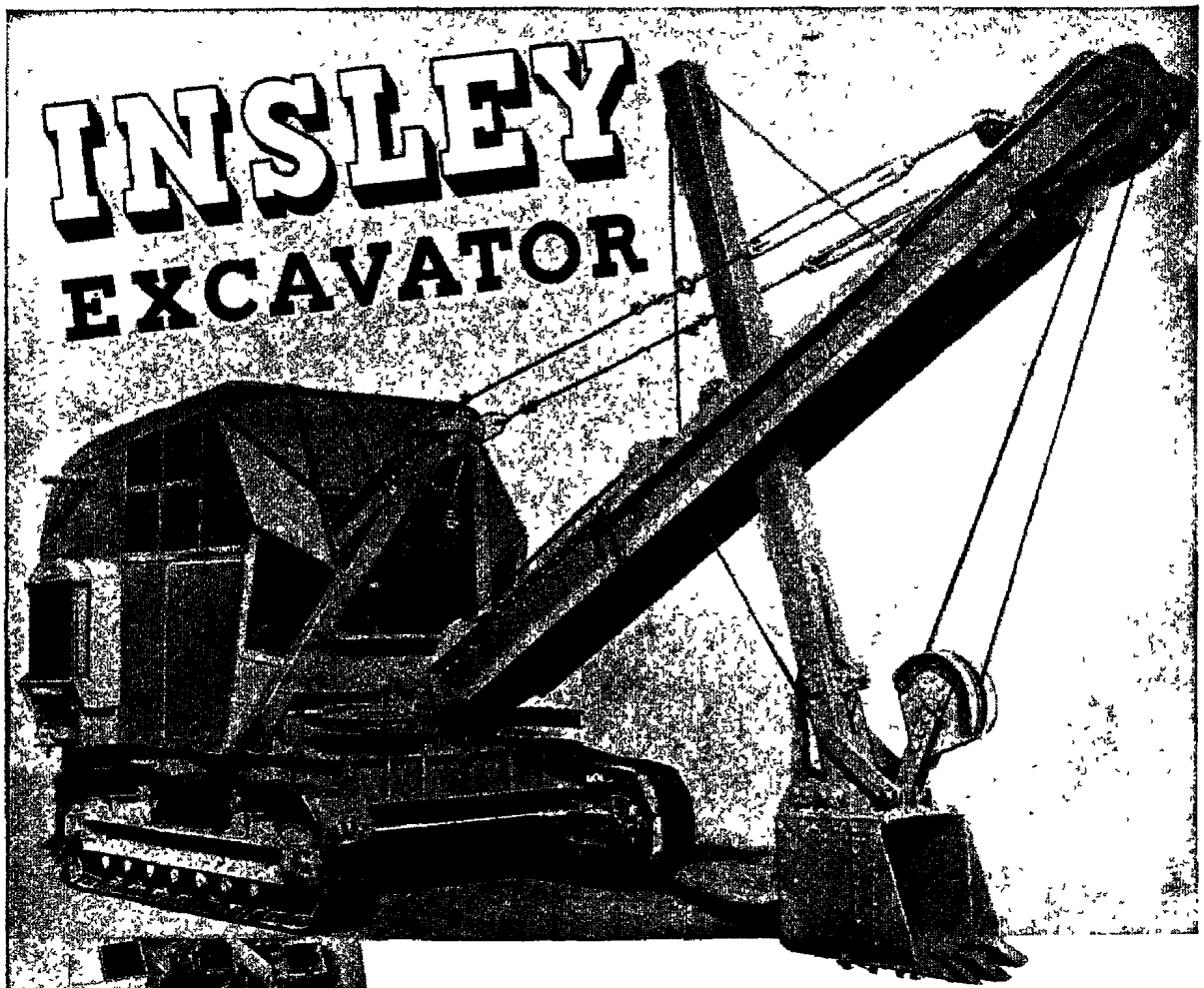
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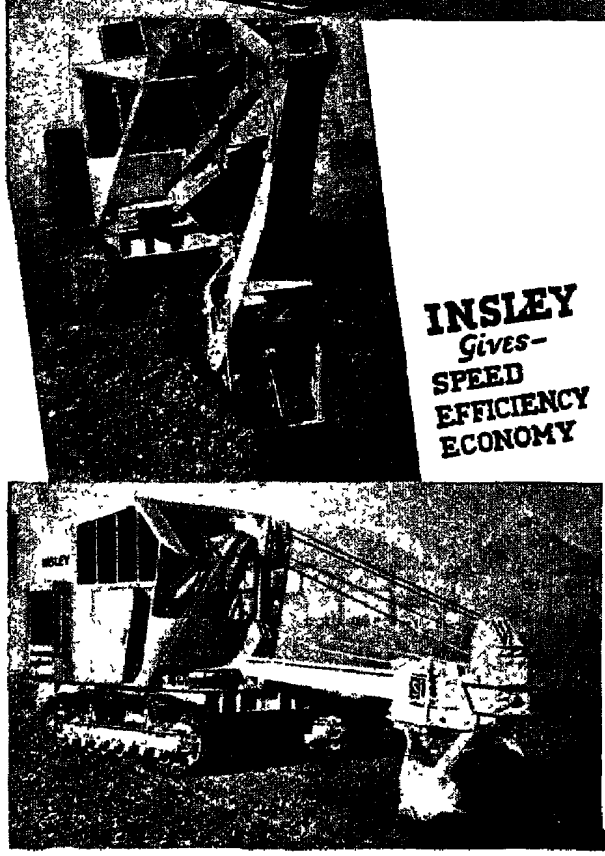
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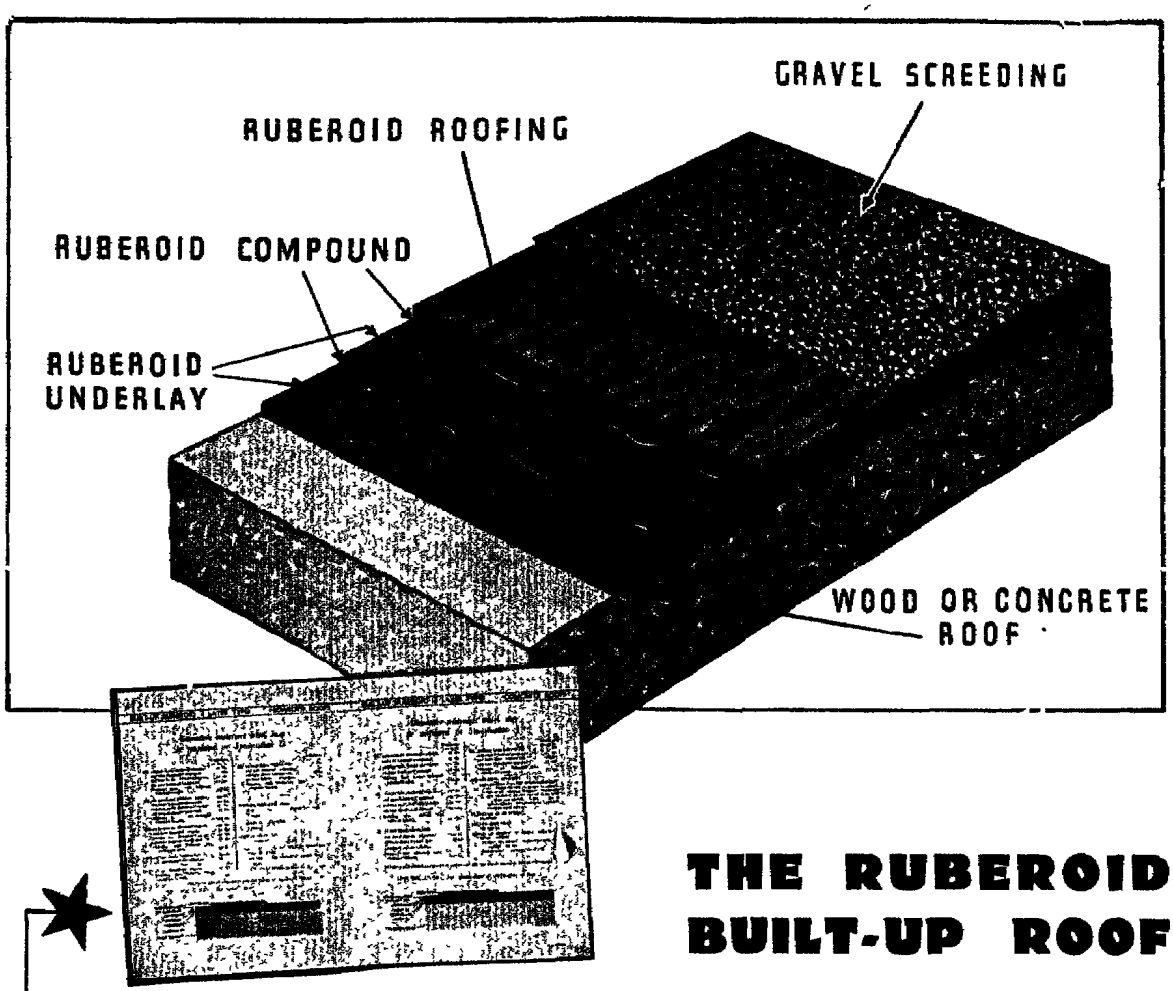
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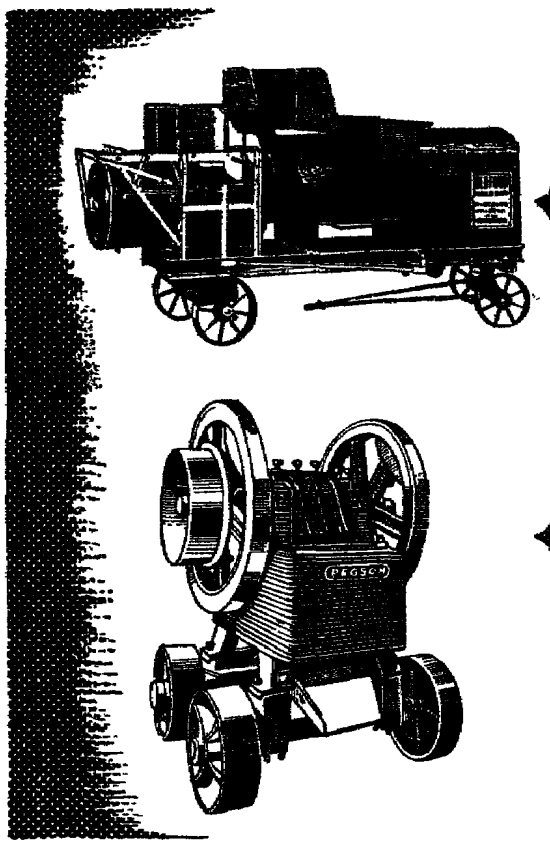
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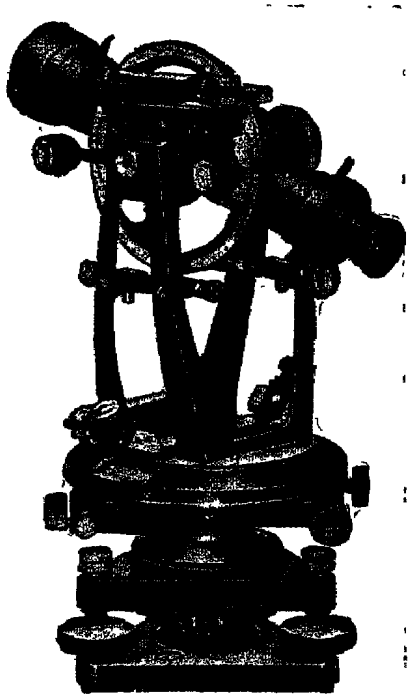
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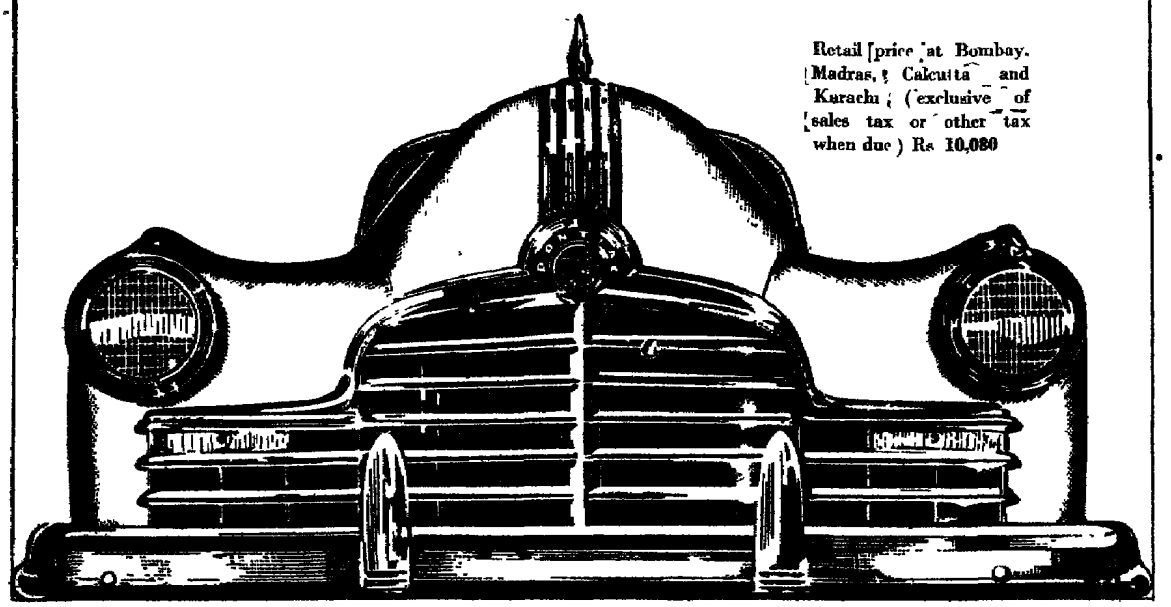
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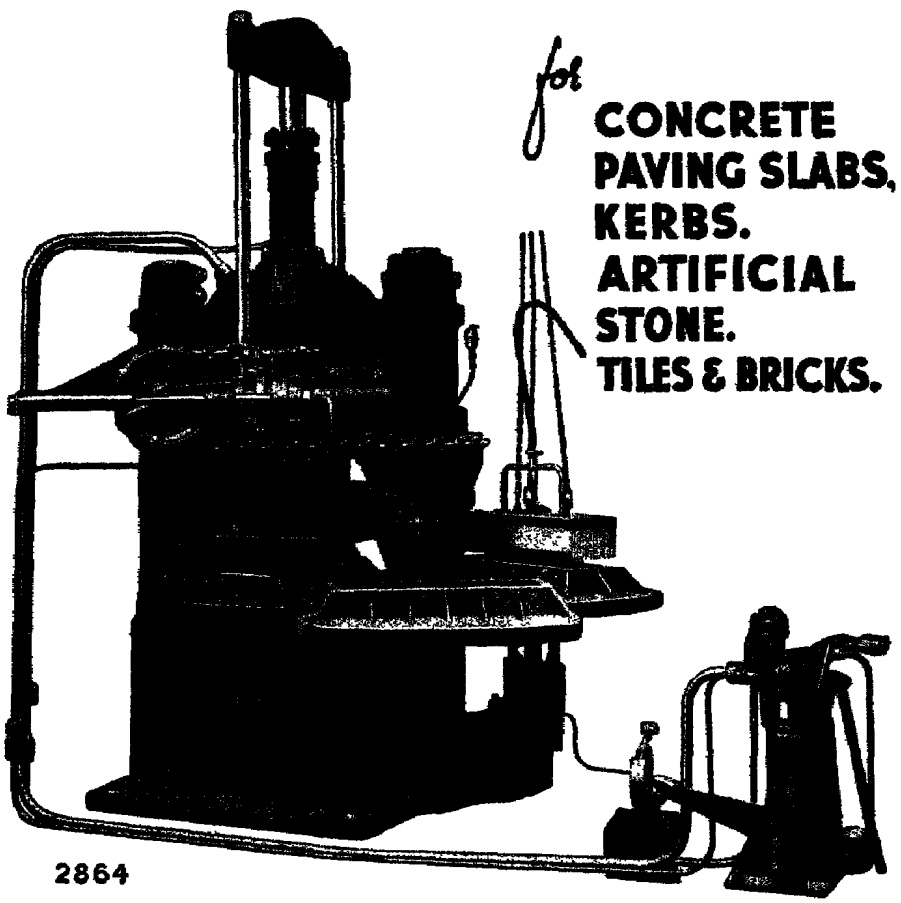
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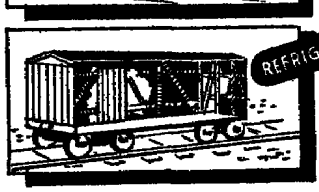
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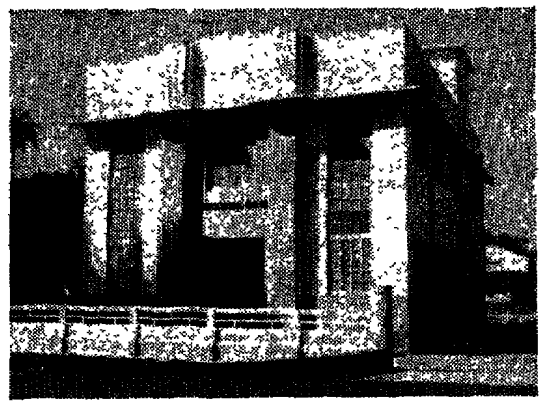
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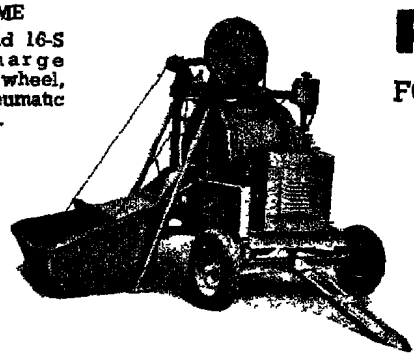
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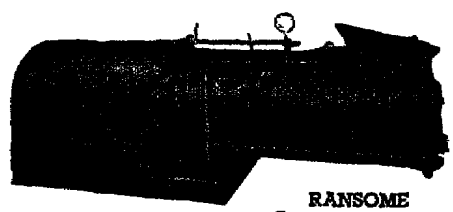
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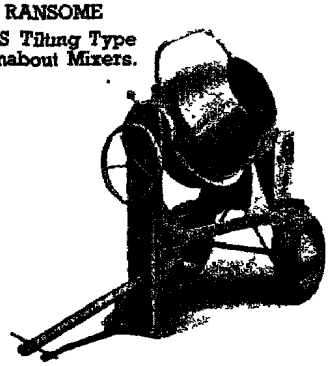
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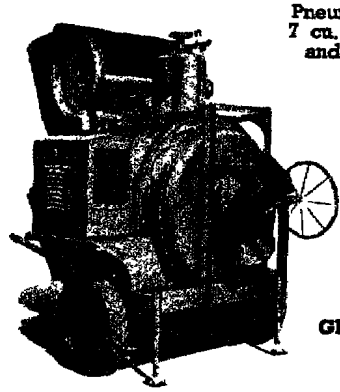


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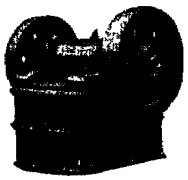
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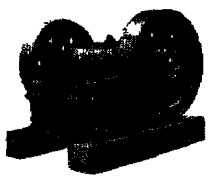
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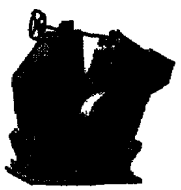


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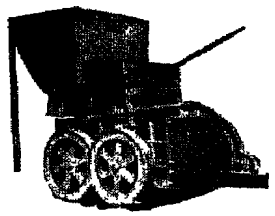
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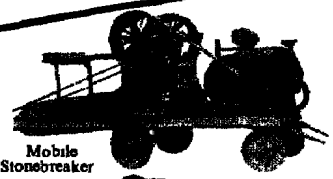


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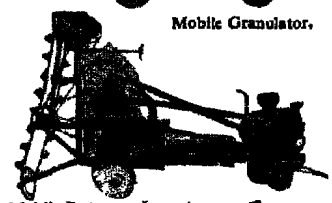
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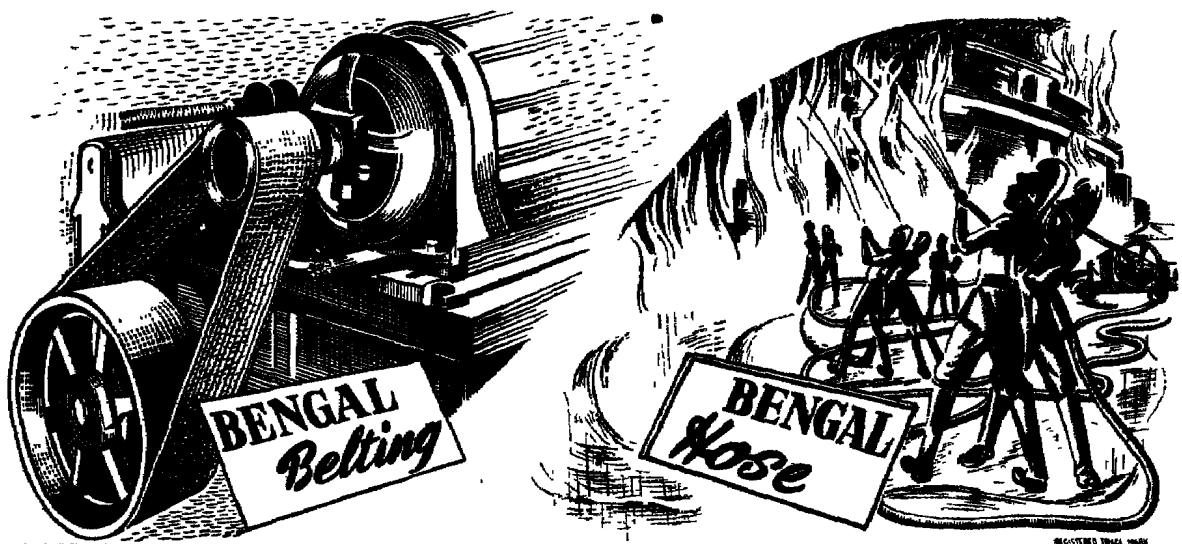


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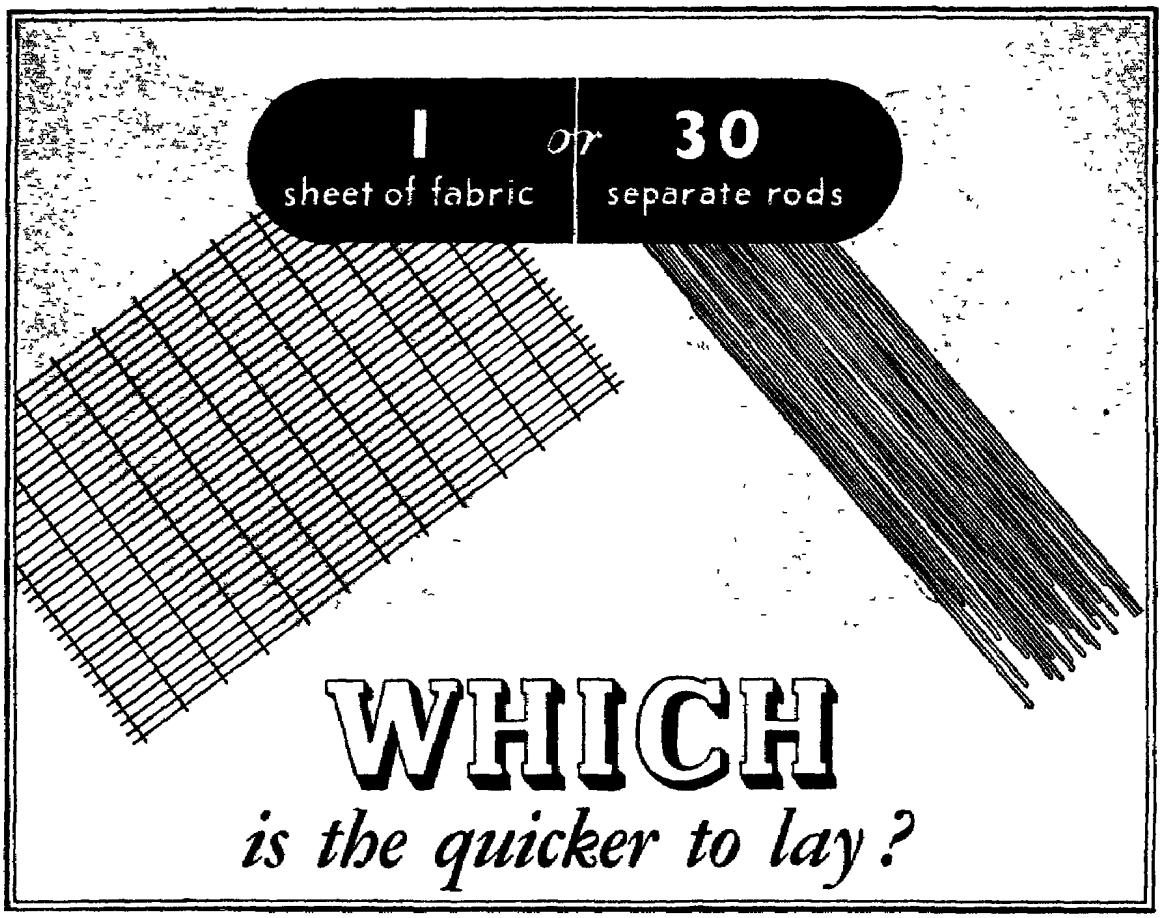


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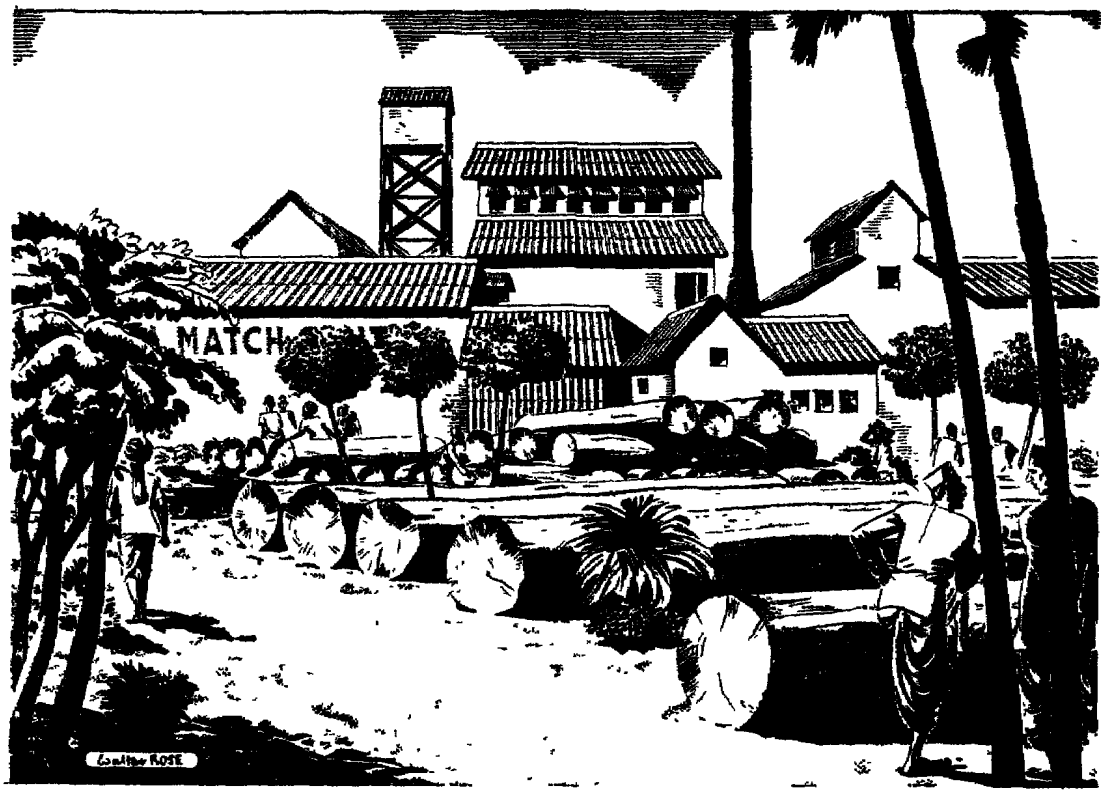
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EDITORIAL NEWS & NOTES



WANTED MORE ROADS

(We reproduce herewith an interesting Editorial comment from the *Bombay Chronicle* on the importance of more and better roads.)

HAS India become sufficiently road-conscious ? This is the question often debated by our civil engineers whose number has happily grown and whose activities now receive better appreciation The rail-road competition, the desire of almost every Provincial Government at present to solve the problem of utilising motor traffic to the best national advantage and the keen public feeling that inter-provincial traffic must increase, are factors which continue to give the transport problems of this country an importance in the public mind which they failed to secure a few years ago.

We welcome, therefore, Mr. Kynnersley's public protest that amidst the number of State-sponsored plans for re-building and reconstruction, the programme envisaged for the road plan by our Chief Engineers only means that " the increase proposed over the next fifteen years is just one mile of road—for an area of eighteen square miles ". It is true and it is equally regrettable that the road plan should, in view of the large size of this country and the urgency of developing adequate means of communication in the countryside, ultimately spell so little of progress. As President of the Indian Roads and Transport Development Association, Mr. Kynnersley in a speech (published elsewhere in this issue), at its annual meeting on October 31 pertinently pointed out that as against the existing road mileage of 300,000, we required a total road mileage of 700,000 even if our villages, some 700,000 in number are to have " on an average no more than one mile of road each to contact with neighbouring villages, markets and railway stations ". It is, indeed, a happy and pleasing sign of the times that from almost every municipal and local board authority in every province in India under Congress Government, a request is made when ministers are on tour, that funds should be spared for them to be spent on road-building and road-improvement purposes. It is not every Provincial Government, however, which can meet their requests satisfactorily unless it has the courage to discover a new taxable source of revenue, call it a road cess to be pooled into a separate Road Development Fund and take care, not to appropriate it afterwards for some other urgent nation-building purpose.

We have to notice with regret that provision of adequate and permanently useful roads in the districts is not yet regarded by many as an essential item in the nation-building programme. We are suffering from the bureaucratic sins of the past The public has been accustomed to believe that a countryside is ready to be developed for trade and industry when a railroad is built through it. It did not obtrude itself easily on public notice that our trunk railways were designed not so much to help inter-provincial trade as to assist the exporters of India's raw materials. It took a long time before feeder railways were built to tap the rural districts. As the situation stands at present, these feeder railways are far too few to develop internal trade. Road-making, had it been undertaken a generation ago, extensively and with a co-ordinated plan in our country, would have to-day, with the advent of the motor lorry, saved many a rural part from famine, as food can be easily reached to famishing areas if only good, motorable roads exist in them. Rural areas, judiciously provided with roads, would have developed market towns and enabled the peasantry to sell their produce quickly. We would suggest this is a problem which the Interim Government assisted by the more enterprising provincial governments should take up in hand as one of primary importance and refer it after short and quick deliberation to a body of expert engineers for execution within a specified period of years. Enough spade work regarding a study of it and its ultimate effect on the creation of wealth and happiness has been done. There is absolutely no necessity of further committees or palavers between the Central and Provincial authorities What is required is concerted action and quick execution.

* * * *

CONCRETE ROAD, NOTTINGHAM, ENGLAND.

Important Experiment at Oxton

Faced with the urgent need of the rapid development of Britain's road system, the Ministry of Transport is co-operating with the Notts County Council in establishing an experimental concrete road on the Oxton by-pass, five miles out of Nottingham.

Work is at present in progress on that part of the by-pass between Oxton village and the main Nottingham-Ollerton road, on which operations had been suspended owing to the war.

Special compacting machinery is being used on a 1,300 yards stretch of road, and concrete slabs of varying thickness and types are brought into operation.

The County Council is carrying out the work, but the Ministry have a watching brief and the representatives of the research laboratory are taking tests and accumulating data.

The work is being carried out by direct labour, and 30 men are engaged on the scheme The concreting stage was reached last week, and the new road will be completed in about three months.

The Ministry have indicated that a 75 per cent. grant will be issued in respect of the portion of the cost, representing normal type of construction, and that they will bear the whole additional cost due to experimental work.—(*Nottingham Guardian*).

THE INDIAN ROADS CONGRESS

The following is the Presidential address of Mr L. 4 Freak at the 27th Council Meeting of The Indian Roads Congress held at Karachi on Wednesday, the 16th of October 1946

IT is my privilege today to welcome Your Excellency on behalf of the Indian Roads Congress and to express our warmest thanks for the honour you have done us in coming here in the midst of your heavy duties and great responsibilities, to open this 27th Council Meeting We greatly prize this recognition of the Indian Roads Congress and I assure Your Excellency that your presence is a very real encouragement to all of us The remark in your speech of the 25th May to the Sind Branch of the Indian Roads and Transport Development Association " It is now recognised—though perhaps not as widely as it should be—that good roads are absolutely essential to every form of development " shows your keen interest in roads and your appreciation of the part roads play and will continue to play in the development of the country

Transport Communications

We live in epoch making times when we have every encouragement to make use of vision, to look ahead, to live rather than to exist, to plan for the future rather than for the present From all sides—H E the Viceroy, Yourself Sir, the reports of various Committees and the technical experts comes the recognition of the need for more and better roads The aim is the transport of goods and passengers goods from source to market, source to fabrication centre, factory to market and so on, passengers between homes and business or on pleasure The objects are —

(*a*) to improve condition of rural life,
(*b*) to assist agriculture of higher standards,
(*c*) to assist development of industries, and
(*d*) to ensure the rapid supply from rural areas to

urban districts of such essentials as milk and fresh vegetables The means for these objects lie in the roads and in the vehicles that ply on the roads Our desire is to ensure that the facilities offered by roads are always one step ahead of the requirements of the user This objective is summed up in the expression 'Transport Communications" – a term suggested for inclusion in one of the schemes to which the Indian Roads Congress has given birth, namely a scheme for co-ordination of all forms of transport

Bullock Carts

I must, however, admit that the bullock cart is not included in this co-ordination scheme but that does not mean that it has been over-looked In its Bullock cart Sub-committee the Indian Roads Congress has a live body of Members who continue to devote much of their spare time to the study of ways and means of improving the cart to the benefit of the Highway authority, the cartman and not the least, the bullocks.

Bhore Committee Report

We realise the need to apply our endeavours as much to the village and minor roads as to those roads less parochial in importance In this connection I may be permitted to quote from the recently published report of the Health Survey and Development Committee This, known more widely as the Bhore Committee Report, states .

" We must emphasise the vital importance of developing village communications in order to enable the health organization to offer efficient service to the people. Without such development our whole plan for the rural area may either be paralysed or lose the greater part of its effectiveness Further the economic welfare of the village population largely depends on the development of rural communications and we stress the need for giving high priority to such development

Soil Stabilised Roads

Thinking of village roads our minds turn to soil and thence to that very important subject of research and experimentation, namely, soil mechanics For some years now the Indian Roads Congress through its willing helpers has taken an active interest in this vital science realising full well its importance in producing better rural roads at low cost. The enthusiasm of these helpers has reacted on the Government of India who are now giving financial assistance in furthering this research and in providing the training in this subject to chosen representatives of all Provinces Already over 30 miles of soil stabilised roads have been constructed and the cost of their maintenance and their life are now being closely observed under traffic during all seasons of the year

Practical Training Abroad

Training in research, application, modern methods of design and construction, administration and the use of mechanical equipment is most essential Again as a result of the recommendation made by the Indian Roads Congress, this year saw the departure to the United Kingdom and thence to the United States of America of the first Government sponsored deputation of Indian Road Engineers The party of 24 is now in America which they expect to leave in November.' We shall await the return of our delegates with great interest Plans for sending another deputation abroad in 1947 are being considered by the Transport Department of the Government of India

Road Rollers

The Members of this Roads Congress have pressed demands for road rollers with the result that the Government of India has sponsored the purchase on behalf of the Provinces and States of 2,267 rollers towards their actual and anticipated demands Of this total 1,548 are to be manufactured in India wholly or in part. It is hoped and expected that the setting up of roller manufacture in this country will result in India becoming fully self supporting in meeting future demands for new rollers and spare parts

Road Lands Enquiry Committee

We welcome too the publication of the report of the Road Lands Enquiry Committee—another of the organizations set up for enquiry purposes on suggestions emanating initially from the Indian Roads Congress

Ex-servicemen

One of the problems we have continually in mind and which we discussed at the last Council Meeting is to provide employment for such ex-servicemen who desire to work on road and other public works. Our considerations have taken many forms and experiments are continuing in a number of different lines Co-operative workshops have been started in Madras Madras, the Punjab and probably other Provinces are trying out Co-operative Construction Units The U P Government is experimenting with a Group Employment Scheme while Bengal, I understand, has formed one or perhaps two battalions of organised labour under their P.W D

Our chief concerns have been to ensure a fair wage and freedom from exploitation Fair wages should of course be minimum wages that still leave scope whereby hard and efficient workers may earn higher wages on task or piece work

Labour Magna Charta

The Central P W.D have introduced a fair wage clause into their agreements and this has been circulated to all Provinces for their consideration and possible application Further consideration has shown that protection is needed for *all* types of labour—not only for Ex-servicemen and not only for Government employees The Government of India have subsequently drafted the Minimum Wage Bill which is still under consideration It may be called " an overdue Magna Charta for Indian labour ". As this bill may become applicable to labour throughout British India employed by Government and by private concerns, firms and companies, etc, for certain types of employment, it has a far wider scope than the Fair Wage clause suggested for inclusion in Government Contract agreements

Irrespective of what form of protection is finally accepted for labour, we Road Engineers wish to devote the greatest share of our time to engineering and for this reason I suggest that some Labour Tribunal Organization be immediately set up for watching the interests and welfare of labour and for dealing with any disputes which may arise Labour disputes should not be an added burden on the shoulders of the engineers

American Experts

Since this Council last met India has had the privilege of a visit by Two

American Engineers of eminence and world repute. I refer to Major-General Fleming, Administrator, Federal Works Agency and Mr Thomas H MacDonald, Commissioner, Public Roads Administration, U S A. Many of us assembled here had the pleasure of meeting them and getting their first hand opinions and advice. The publication of their report will be awaited with the keenest interest by us all but I may be forgiven if in the remarks that follow I make mention of what, from their speeches and conversations I understand to be their considered views on certain road matters with which we in India are essentially concerned.

Road Construction Technique

It is gratifying that the American Experts had no serious technical criticism to make on our present road practice and design, on the contrary, they recorded that there was much to praise

They, however, pointed out the need for a far greater interest in the "Geometric" design of Highways or what we generally term "layout" In particular they advised that much will be saved if our roads are planned now with a suitable geometric design which includes location, alignments, curves, superelevation, transition curves, intersections, sight distances (vertical and horizontal) and width of right of way Strength of construction can be upgraded as traffic increases whereas the improvement of the geometric design is often impossible without relocation, because of the high cost of acquiring adjacent property.

Road-Rail Competition

May I now refer to the vexed question of road-rail competition. For a long time past, we have felt our road expansion programme being throttled by the influence of railway interests. We feel that there is ample need for both roads and railways and so road construction does not need to be held up to ensure the finances of the railways So far we have not always succeeded in getting the Government of India to share this view but we hope that the dawn is breaking on this impasse. In brief, instead of road-rail competition I suggest there is, by co-ordination, a place for both roads and railways. Permit me to quote an excerpt from the speech delivered by Mr D'Alton C. Coleman, Chairman of the Canadian Pacific Railway Coy, on May the 1st, 1946, at the Annual general meeting of the shareholders —

> "*The railways do not covet a monopoly; they recognise that buses and trucks and steamships and airplanes have a right to the traffic which they can handle economically and efficiently The need is therefore evident for equitable regulation and for the re-establishment of genuinely competitive conditions so that each factor in the transportation picture may operate on the basis of sound economic principles. In no other way can the nation be supplied with adequate transportation at minimum cost to the public.*"

A particular note may be taken of the object mentioned, namely, adequate transportation at minimum cost and we might well add minimum risk of damage and in the case of passengers maximum comfort and convenience

The American experts told us how impressed they were and are by the importance ascribed to statistics in U S.A. The daily average number of motor vehicles making the transcontinental run on the excellent Highways stretching from the Atlantic to the Pacific amounts to less than 400, a negligible figure when compared with 32 million, the gross number of registered vehicles at the time the count was made This clearly demonstrates that commercial transport operates within a relatively short radius. For instance fully 85% of such transport in the United States moves within a radius of 20 miles and so, particularly in the movement of goods, relieves railways of expensive short hauls which are often done at a loss. The railways still continue to predominate in long distance haulage. In America railways are abandoning their short branch lines—which even before the advent of good highways were operated at a loss—and at the same time are investing their funds in buses and lorries to move passengers and freight short distances and at a profit. The American experts noted an apprehension that the development of a comprehensive highway system in India will ruin the railways which are a source of considerable revenue to the Centre. They believe these fears to be unfounded basing their opinion on the experience in U S A as ascertained from carefully compiled statistics.

I have given the view points of a large railway administration and of these American experts who have been dealing with roads in U.S A. on a grand scale. I think, therefore, we can accept such views and continue to press on Government to adopt a sympathetic attitude towards the goal of obtaining the best form of transport for India—by road transport where that is more suitable and by railway transport where that is more economical or convenient.

Economic Planning and Research

I have already referred to the question of research I should like now with your permission to amplify my remarks by referring to the more comprehensive subject of economic planning and research. Here again India can benefit greatly from the recognition that has been accorded to this vital matter in the United States. I cannot do better than to quote the remarks of our expert American visitors:

"Perhaps the most important single element of economic value that has been introduced into the highway administrative practice in recent years in the United States is the provision for planning. A small percentage, namely 1½ per cent. of the total Federal and State Co-operative funds for highway construction and improvements is dedicated to planning This fund may be used for *no* other purpose than economic planning, research and overall study of projects to be undertaken in future years. It is now more than 10 years since this fund was first dedicated to the purpose of planning and in every state there have been, and there are continuing, surveys of the economic facts which contribute to the formation of a sound improvement plan. On the basis of the accumulated information painstakingly acquired through years by these planning operations, perhaps more detailed information exists upon public roads, their current physical status, and their future needs, than exists for any other public undertaking.

This information, now carefully analysed and summarised in usable form, is constantly used by the State and Federal Highway administrative officials and becomes the supporting evidence upon which legislative bodies, both State and Federal, determine their policies and the annual amount of funds to be made available. This approach to a determination of a sound public policy for public highways has shown the fallacy of short term programme and the importance of an expenditure programme extending over a series of years *to make possible economy and efficiency.*"

The consequence of the 1½% provision from road funds has been that the American Federal Government and each State in the U S A, have set up organizations to handle economic planning and research and, thanks to assured finances, continued employment has enabled efficient staffs to be recruited The knowledge gained has been invaluable and has put road planning questions out of the sphere of politics and personal opinions Irrefutable facts can be presented to the Federal or State Government.

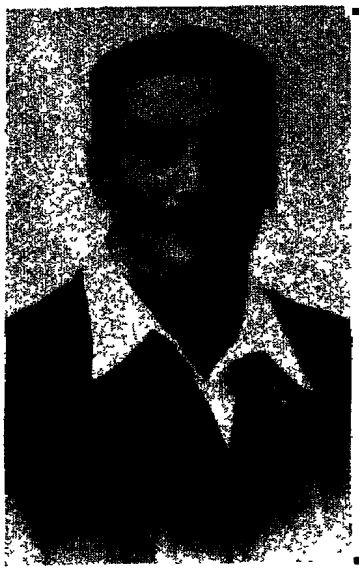
The American experts from discussions with scientists and engineers in India formed the opinion that because of undue restrictions on every expenditure the latter are experiencing considerable difficulty in being able wisely to spend the money which is allotted to them. The experts advise that considerable amount of latitude in the expenditure of funds on research and planning should be allowed to the scientists and engineers, as no one can predict the exact directions in which the research will go and incidental expenses therefore cannot be accurately predicted

We have not been entirely starved for funds for research and experimental works as small sums are made available for this purpose from the reserve in the Central Road Fund and this year the Centre has a specific budget provision under the head "Capital Outlay on Development-Communications", from which approved schemes of this nature are being financed. But we are only just awakening to the need for economic planning. Funds for employing staff to collect statistics, etc, towards economic planning of our roads have still to be arranged by the Provinces while the Government of India has only recently recruited staff for this purpose.

Assured Finances

Mr Vesugar in the Lucknow Council Meeting very correctly pointed out that for Post-War Road Development "financing our plans is the last hurdle".

The American experts have expressed the opinion that a really adequate system of highways will never be possible of attainment under the system of financing



Obituary

MR. D. K. KAPADIA

It is with great regret that we record the death on 30th September of Mr. D. K. Kapadia, a Senior Assistant of The Cement Marketing Co. of India, Ltd., due to heart-failure at the age of 57.

After graduating from the Elphinstone College, Bombay, he joined The Cement Sales Department of Messrs. Killick Nixon & Co. in 1923 and was transferred to The Cement Marketing Co. of India, Ltd. on its formation in 1930 where he gradually rose to the position of a Senior Assistant in charge of sales in Western India.

"Dolly," as he was known among the cricket-loving public, took a leading part in this game for over a quarter of a century, and only a few days before his death he was nominated to the all-India Selection Committee of the Board of Control for Cricket.

By his death Bombay has lost a very popular cricketing personality and The Cement Marketing Co. an able and popular officer.

and administration that has prevailed in India in the past. In America the Congress, in 1944, enacted a law authorising a 3 billion dollar post-war programme over a 4-year period for the construction, improvement and replacement of highways. This law embodied the most comprehensive highway programme which has ever been produced in America.

We are convinced that a similar law providing annual non-lapsing funds for highways would prove most beneficial to India. Every agriculturist knows that a good crop cannot be achieved without adequate and regular waterings. Similarly, we road engineers cannot provide the roads this country needs unless we can be assured of adequate and regular budget provision coming forward year by year. It seems fundamental that Government should hypothecate certain taxes solely for financing roads and the obvious funds, for such dedication to road finances, are import duties on motor vehicles and spare parts, petrol tax, sale tax on petrol and motor vehicle taxes. I have already mentioned the Road Fund financed by an extra tax on petrol as an existing example of a tax hypothecated solely for financing road schemes.

Unfortunately petrol is still very much controlled in this country thus throttling motor transport development and petrol tax receipts. Hence the money which should be available for financing roads is at present being throttled but it is hoped petrol control will soon be unnecessary. In U.S.A. the Federal Government is committed to grant the States under it a very large amount during the next few years for road construction and in order to finance road construction programme the trend is towards increasing the tax on petrol. Already 24 States have increased their petrol taxes and it is expected that by next year 45 States will have taken such action. India, its Provinces and States can do what America and its States have done. If the existing petrol tax is insufficient to finance our road construction programmes then the question of increasing the tax might well be considered. Other sources of finance for our road programmes should also be explored as assured finances are a *sina que non* for well schemed road development in this country

The present system in India whereby funds have to be spent by the end of March or otherwise lapse is both wasteful and counter to proper development Comparison between the operation of the Central Road Fund and that of the ordinary Provincial budgets demonstrates the difference in effect and control.

Congress on Road Development

In spite of these financial difficulties but thanks to the initiative created by the Nagpur Report we have on the whole made satisfactory progress in our road development planning and construction We now possess the Centre's approval to monetary limits for Provincial road improvement programmes for execution up to March 1952. The five year programme for Central liability on National Highways is tentatively at the figure of Rs 30 crores for works of development, Rs 8 crores for maintenance and a further sum for financing research and the setting up of research organizations, etc. In the Provinces the approved programmes of road development total above Rs 150 crores towards which amount the Centre is expected to contribute liberally. The amounts earmarked for expenditure on the development of roads average about 20% of the total amount of the development programmes throughout the country. This is not unsatisfactory but for the uncertainty in regard to regular finances. The tacit acceptance of a paper programme can only be translated into actual road development if the finances are assured.

Propaganda

Many years ago Mr. MacDonald to whom I have previously referred mentioned that we pay for the roads whether we have them or not but we pay more for them if we do not have them. In 1944 at the Bombay Council Meeting Mr. Vesugar referred to the practice to date to pay for road improvement and construction out of current revenue as a "pay as you go" policy I am afraid that in the years immediately preceding the war many of the road tax payers were of the opinion that the amount spent on roads was less than the income from import duties on motor vehicles, petrol tax, sale tax on petrol, motor vehicles tax, etc., and so the position might even be termed "pay as you do not go."

As the future Government in India at the moment is in the progress of being moulded we might temporarily ease up on our efforts urging assured finances but in the meantime I would venture to suggest that we start propaganda with the view to get the road using public to fight our case for us on the lines that the public, who pay taxation towards the roads in the form of import duties and taxes already mentioned should demand the roads for which they have paid such import duties and taxes. Once we can succeed by propaganda or any other means in getting the public to act for us in this way than the Legislators, whoever they may be, must listen to that public demand for the assured financing of the roads that India needs. A small sub-committee of this Congress has already been suggested to examine the available means of publicity, official and non-official, and the possibilities of increasing the education of the public in road matters. May I go further and suggest that in addition to the issues mentioned above, this sub-committee should by propaganda educate the road tax payers to demand value in the form of adequate roads for the taxes they have paid. Up to date some Legislators and administrators in India have stated that it is contrary to the cannon of financial propriety to hypothecate certain taxes for any special purpose though there is at least one existing example already mentioned. If the road tax payers are once educated by propaganda they should be able to exert such pressure that road, motor vehicle and petrol taxes will be solely hypothecated to road finances and thus the regular development of roads in India will be assured.

INTEGRAL WATERPROOFING OF CONCRETE STRUCTURES

By L. E. HUNTER, M.Sc., Assoc.M.Inst.C.E., Assoc.M.I.Struct.E.

THERE are three forms of integral Cement waterproofers available, *viz* Pastes of various specific gravity and colour, powders of different specific gravity and colour, and fluids of widely differing viscosity and specific gravity and colour

The essential difference of these three types is that of mixing Usually, powders and pastes require considerably more effort to mix Usually pastes are very dense, and since they have to be worked by hand into the condition of a thick cream, free from lumps before adding the final amount of water, it is only possible to mix a very small proportion at a time. All lumps must be eliminated, otherwise trouble is certain Should there be a failure to mix the paste free from lumps, these having a dry core, cause considerable difficulty, especially in rendering, because they invariably initiate cracks and crazing. If the paste is used in a lumpy condition, then there is a complete waste of a corresponding amount of material. After the last of the water used for mixing the paste has been added, there is a tendency for separation to take place, with the result that the mixture in the upper part of the container tends to lose its density. Therefore, with pastes, it is necessary that the mixture is stirred well before each addition to the cement, otherwise the benefit of the waterproofing qualities of the paste will be partially lost. Most of the pastes are dense and of a glutinous nature, and are delivered in barrels or metal containers.

Powder Admixtures

Powders are often mixed neat with the cement, and never with the gauging water before the fine and coarse aggregates are added These are known as admixtures Sometimes this term is used for all integral waterproofers Some powders are added at the works with the cement This has the advantage that correct proportions of powder are obtained This is not always possible under the variable conditions in which site operations are undertaken Where powders are mixed at the site, the general procedure is roughly as follows. The cement to be used is first spread over a flat surface to a depth of one inch or so The waterproofing powder has then to be sprinkled evenly on top of the cement in the required proportion The cement and powder are then well mixed and are afterwards screened two or three times before adding sand or ballast Since powders are usually light, it is necessary to protect the mixing operation from the atmosphere Unless extreme care is taken over this process, the waterproofing effect of the powder on the resulting concrete will be haphazard and unreliable It is believed that the powder particles swell and harden after coming into contact with the lime contents of the cement. The swelling particles fill the voids in the mix as it sets and dries out This result is not always realised in practical construction, due mainly to the site conditions of mixing With waterproofed cements this has more likelihood of realisation The latter is a far better product than admixtures since the proportions are strictly controlled The cement itself is actually improved in performance, since by its use an increase in strength in the concrete is obtained over the use of ordinary Portland Normally, the cement is manufactured as ordinary Portland cement up to the stage of grinding. At this point, another material is substituted

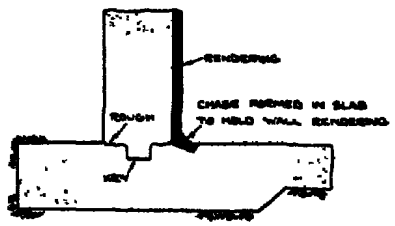


Fig. 1. Construction joint at base of mass concrete wall.

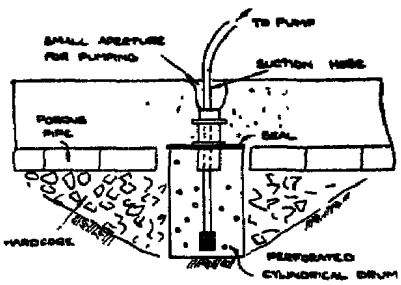


Fig 2. Typical sump and suction pipe.

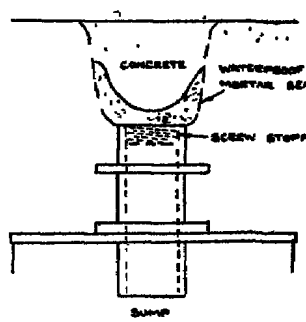


Fig 3 Sealing process to top of sump

for a percentage of the raw gypsum which is often added at the final grinding This is a processed gypsum which helps to increase the hydration of the aluminates and silicates present in the cement.

The advantage of having a waterproofing medium present in the cement is considerable. The worry and responsibility of supervising and providing adequate supervision for site admixing is abolished, and the correct proportion of waterproofer goes into the concrete evenly

The waterproofed cement is more resistant to the corrosive actions of acids, alkalies, sewer gases and soluble harmful salts so often present in industrial waters than in ordinary cement.

Liquid Waterproofer

With the use of a liquid waterproofer, the filling of the voids is more completely realised than by using powders or pastes. The action of a liquid waterproofer is distributed throughout the mix, thus setting up a uniform chemical reaction on the particles of the cement, thereby filling all the voids, and consequently adding to the binding properties of the cement itself When the solution comes into contact with the cement matrix it starts to crystallise and thicken. This process continues until the cement has completely set and all traces of colouration has disappeared A particle of cement can be considered as a latent life which can only be activated by the addition of water, which causes a chemical reaction The result of this reaction is that the cement binds itself firmly to the particles of sand and ballast with which it is in contact The usual period of time necessary for the completion of this chemical reaction is from 12 to 18 hours at a temperature of 60° F to 65° F, and if the water present is in the exactly required proportion the total and perfect setting of the cement is obtained More water than is required is often present, and some of this excess tends to accumulate in any voids present and thus weakens the concrete itself The use of a liquid waterproofer tends to thicken and produce a fatty matrix and helps to retain the gauging water by its utilisation in completing the chemical reactions of setting. Many of the bad aspects of too short a chemical reaction period are eliminated.

The placing of waterproofed concrete should always be carried on continuously to avoid the weakness of day work joints. If the concrete cannot be completed in the course of the day, it is necessary to leave the edge as rough as possible and on the resumption of work to adequately water the edge and then brush it over with a thick creamy mixture of neat waterproof-cement It is very necessary to follow on quickly with waterproofed concrete before the grout dries If the concrete does not contain reinforcement it is necessary to embed a strip of expanded metal or other reinforcement across the joint so that half of the strip will protrude and form a bond when it is covered by the abutting layer of concrete. The new concrete should be well tamped round the mesh reinforcement.

Basements

When the walls of a tank or basement in flooded ground are of concrete, especial care should be taken with the joints between the bottom of each wall and the top surface of the concrete floor. The surface of the floor concrete, where it occurs under the walls, should be left as rough as possible, and a rough groove formed for the toe of the chamfer or cove of the interior angle between the wall and the floor (Fig 1). Before the concrete sets, 6 in to 9 in. wide strips of galvanised corrugated iron, sheet tin or other sheeting are forced edgeways into it midway in the thickness of the wall, leaving half the width of the baffle protruding These form a tongue, or " baffle " The surface of the joint in all cases should be well grouted before the wall concrete is poured The thickness of the former should not be less than $\frac{1}{8}$ in

Waterproofing a basement floor is often a difficult problem. Sometimes springs run through or near the site. In these cases, the water must be diverted into a main pipe drainage or series of pipes and hence into the normal drainage system, or by the introduction of a sump and pumping out

Water-logged Ground

When a structure is built in or on water-logged ground, the waterproofed work is subjected to a hydrostatic pressure that may do considerable harm to the floor, although internal renderings may not show signs of damage When floors are subjected to such conditions, the upward hydrostatic pressure is always proportional to the head of water above the soffit to the waterproof concrete, it is usual to design rafts of this nature to fully resist such fluid pressure—which may be very large. When the water pressure is small, the hydrostatic thrust is counterbalanced by the floor load. A nominal amount of mild steel reinforcement is required, if only to provide against cracking caused by slight subsidence or other detrimental effects

When the pressure of water exceeds 1 ft , it is economical to employ a thickness of concrete suitably reinforced to withstand the upward water pressure In this case, the floor is designed as a raft, the whole of the reinforcement being in the top of the slab, except for rods turned down at the perimetral supports for shear and reverse bending moments

A further point is that especially in basement construction, the latter must not be subjected to water pressure until the concrete is hard enough to develop a considerable strength The provision of a sump either externally or inside is necessary to drain off the water If the latter can be carried to the external drainage there is no need for pumping. Continuous pumping from the sump is necessary during the construction of the raft and walls, and frequently the roof as well due to the buoyancy imparted by the water pressure exerted on the structure For ordinary Portland cement not less than seven days should be allowed, and for rapid hardening cement three days before the basement is subjected to water pressure.

Where the basement has a natural run off, such as by means of sloping ground or is adjacent to a river, etc , it may not be necessary to form a sump. Land drains can be incorporated in the subsoil beneath the foundation of the basement Difficulties of access, or obstruction by other buildings or site conditions, sometimes render it impossible to provide a sump on the outside of a basement It is then essential to form the sump in the floor of the basement itself The sump can consist of a small concrete catchpit or stoneware gully. A very economical and effective sump is to use a sound perforated cylinder, say 18 in deep by 12 in diameter This is placed in a hole dug beneath the floor level. The pail

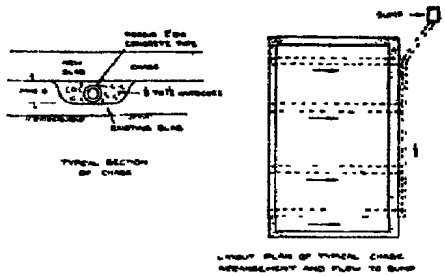


Fig. 4. Typical waterproofed reinforced concrete basement.

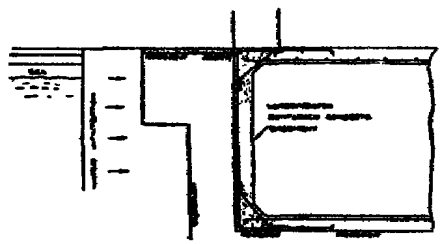


Fig. 6. Drainage to new slabs superimposed on existing concrete by means of chases

itself is surrounded by hardcore, bricks, etc. The access flow should be obtained by 3 in or 4 in diameter land drains or open-jointed spigot and socket stoneware pipes, or alternatively 3 in diameter porous concrete pipes, and all of these should have a surround of 6 in minimum of hardcore The cylinder should be provided with a board or steel cover Precautions should be taken to keep out detritus, silt, etc , from the pump

A double-flanged cast iron tube should be provided of sufficient diameter to allow free passage of the suction pipe and filter The flanges when the tube is encased in concrete provide a watertight joint (Fig 2).

Fig 3 shows a finished raft with the suction pipe removed and the top of the tube sealed by a cap or stopper The former is more convenient for reopening for inspection The hole above the stopper should be sealed off with a rich plug of waterproofed grout and topped with waterproof cement A floor rendering all over is not necessary.

Permeable Subsoil

If the subsoil in which a basement lies is permeable the water can freely percolate through it, and the water will drain from the surrounding ground into the sump. If the building is constructed on rock or impervious subsoil, the water has to be drained from the excavation through weep holes in each of the outer walls. The water is then carried to the sump by means of hardcore or channels placed beneath the floor slab.

For new construction, the method of waterproofing is determined largely by the design, shape and future use of the building and the nature of the hydrostatic conditions imposed The various method may be roughly classified in the following manner.

(1) Tank construction involving monolithic use of reinforced concrete for walls and floor and frequently roof as well, due precautions being taken with joints at wall and floor and roof This is only practicable in new work, but is the most economical for such cases and is the best defence against bad hydrostatic conditions imposed on marine or inland water sites Fig 4 shows a typical construction adjacent to a very porous quay wall.

(2) Reinforced waterproofed concrete floors, with mass concrete walls, but for economy of the more expensive waterproofing cement, with provision of a dual mix, the waterproofed facing being 3 in or so wide on the external face. This construction is generally suitable for deep heads of water and especially for marine work For shrinkage reasons, the walls should be undertaken in short lengths with adequate keying It may be necessary to use a vertical metal baffle at each joint

(3) Monolithic mass concrete construction with floor in mass concrete in addition This method is very suitable, when good rock is the foundation, and as a result a slab of reasonable thickness can be used. The floor should preferably be waterproofed throughout its full depth, but when the rock bed is fairly level, the lower 4 in should suffice for waterproofed concrete. The walls should be externally faced as in (2)

(4) Composite structure of mass or reinforced concrete floor together with brick or stone walls (the roof being in reinforced concrete). This method is not very suitable for large heads of water and should be used for damp conditions only (Fig. 5). In restricted sites, it is not possible to put on an external cement waterproofed rendering. The rendering in this case has to be internal. It is

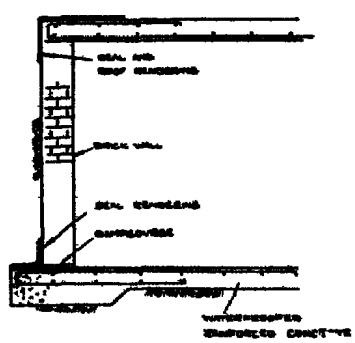


Fig. 5. Composite basement suitable for dampness only.

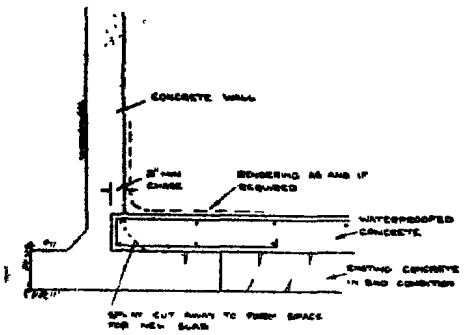


Fig. 7. Typical superimposed raft designed for water pressure Left.

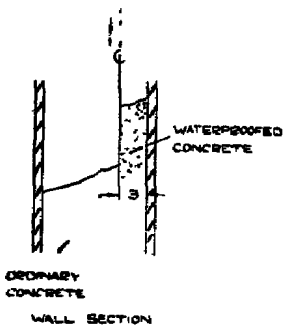


Fig 8 The use of a movable shutter to provide a waterproof wall facing Right.

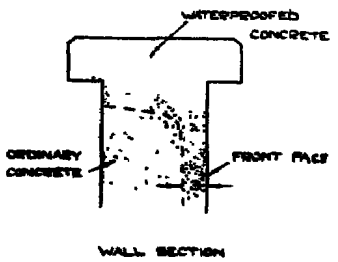


Fig 9. Waterproofing of wall coping.

necessary to waterproof all walls, whether of concrete, brick or masonry if below ground level, unless extremely dry conditions occur and there is a permanent prospect of no water collecting at the site

When new waterproofed concrete is laid on an existing floor surface, it is essential to provide sub-surface drainage to the latter To achieve this it is necessary to hack chases in the top of the old concrete The chases should not be less than 4 in deep with a cross chase leading into the sump Porous concrete pipes or agricultural drains surrounded with stone of ¼ in. to 1½ in grading are necessary. When no sump is provided internally, the chases should extend through the basement walls and connect with drainage channels and pipes leading to the external sump. (Fig 6).

In existing basement waterproofing, it is highly essential to ensure continuity of construction. The preliminary step is to clear the room of all plant, disconnect all piping, and on replacing these items precautions must be taken not to crack the floor and wall surface by dead load or radiant heat. Asbestos sheeting around boilers is a wise precaution against heat, and timber or concrete pads should be provided to withstand the dead load. Under no circumstances should the new work be pierced Brickwork forms a very good key to a cement rendering, but plain stonework does not. All sharp corners should be chamfered, for rendering whilst internal angles are coved

All old concrete surfaces should be well hacked or chipped and swilled with clean water before new concrete is placed. Vertical chases all round the existing brick or concrete walls are necessary in order to provide an adequate seal and frequently a means of ensuring a reaction against hydrostatic pressure (Fig 7)

Roofs

The waterproofing of roofs with integral waterproofers follows very closely that of floor construction. New roofs can be provided with waterproofed cement included or a waterproofing admixture to the mix Old roofs can also be waterproofed by the addition of a top rendering

Roofs waterproofed by an admixture to the concrete mix or by waterproofed cement are constructed in the usual way Where the roof is thick, as for air raid precautions, the upper portion of the roof need only be waterproofed To ensure complete uniformity of adhesion between the bottom layer of non-waterproofed and waterproofed concrete, the latter should be laid within two of three hours after the laying of the bottom concrete Construction joints cannot always be avoided and these lead to trouble unless great precautions are undertaken This reason gives the surface waterproofer an advantage over the integral waterproofer, since with the adoption of the former, the number of joints do not matter Top screeds are also good, but are not so positive nor yielding as surface waterproofers such as membranes or bituminous compounds.

Walls

Since walls are the portion of a building which is mostly exposed to driving rain

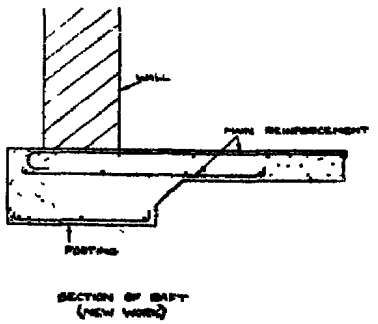


Fig 10. Typical raft designed for water pressure

and creeping dampness from the foundation, it is necessary to procure the maximum impermeability possible with the available materials and labour Whether waterproofing cement or an admixture is used or not, it is vitally necessary to provide efficient joints The importance of this point cannot be too strongly emphasised The troubles ensuing from poor horizontal wall joints will often occur only after a considerable period. Weathering at each joint should be provided The rain should have to drive uphill to gain access into the structure

All laitance and dirt must be cleaned off before new construction commences Wire brushing, water under pressure from a hose pipe or other means should be employed to clean the old joint, and due care is necessary to ensure that the ensuing lift of shuttering should be tight

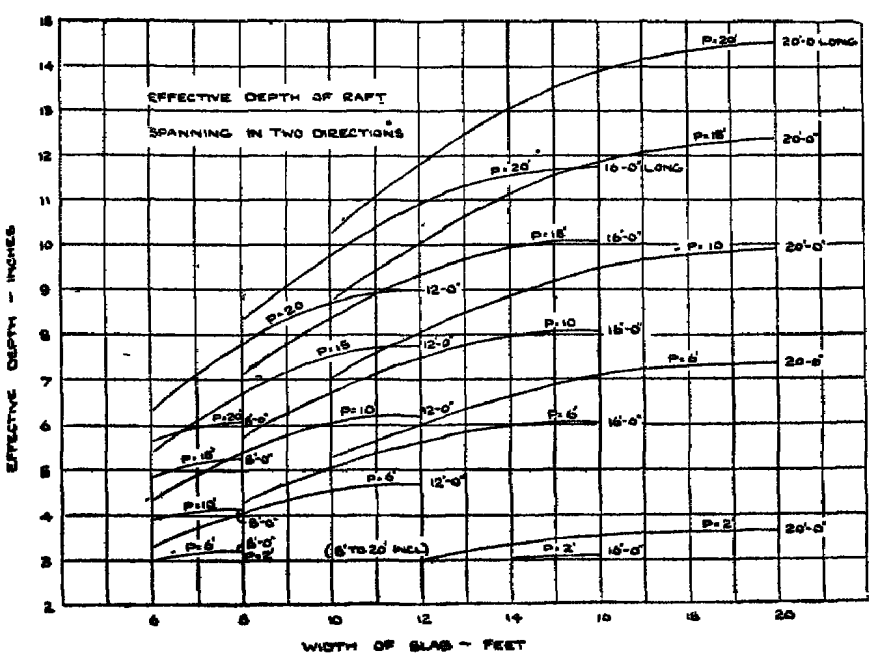


Fig 11. Required thickness of rafts.

to avoid loss of water and cement and thus consequent stoniness, if not bulging

Number of Joints

The number of construction joints should be kept to the bare minimum, since these are future weak places in the structure and may give trouble unless carefully constructed. Thick walls for economical reasons need not be constructed for their full width in waterproofed concrete Fig 8 is an illustration of the method of using only a facing of waterproofed concrete from 3 in to 4 in thick Walls of thickness greater than 11 in, or 12 in can be economically provided with an inside shutter. This is usually placed on the inside face of the reinforcement and withdrawn as concreting progresses, in order to obtain mixing of the adjoining concrete before the initial set occurs and to form a waterproof key The removal of the shutter immediately after tamping or vibration has ceased, permits the waterproofed and non-waterproofed concretes to coalesce freely and form a homogeneous wall The non-waterproofed concrete need not be of the same strength as the waterproofed portion, if the wall is not reinforced and has to withstand substantial load Only such strength as is necessary for flexural or direct load reasons is necessary. It is advisable to use a very well-graded mix for the facing concrete to ensure maximum impermeability.

Such facings to retaining walls and wing walls of bridges would greatly enhance their appearance, besides increasing the waterproofing capabilities of the members A facing of waterproofed concrete is considerably brighter and has a better appearance, resembling stonework more than does un-waterproofed cement. Fig 9 denotes a derivation from the method of waterproofing thick walls. This illustration shows the method applied to a wing wall of a bridge with the coping waterproofed in addition to the front face It frequently happens that disintegration of wing and retaining walls occurs at the top coating. This portion of the wall is subjected to greater atmospheric variation than any other part of the wall Once the coping is in a bad condition of deterioration, weather can easily attack the top of the wall and complete disintegration may follow as a result

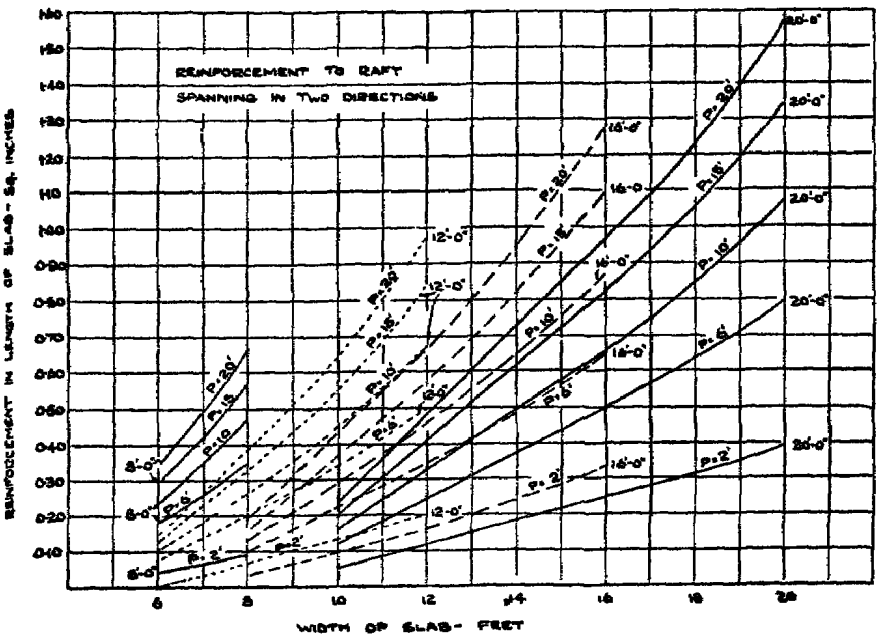


Fig 12. Steel reinforcement required in rafts.

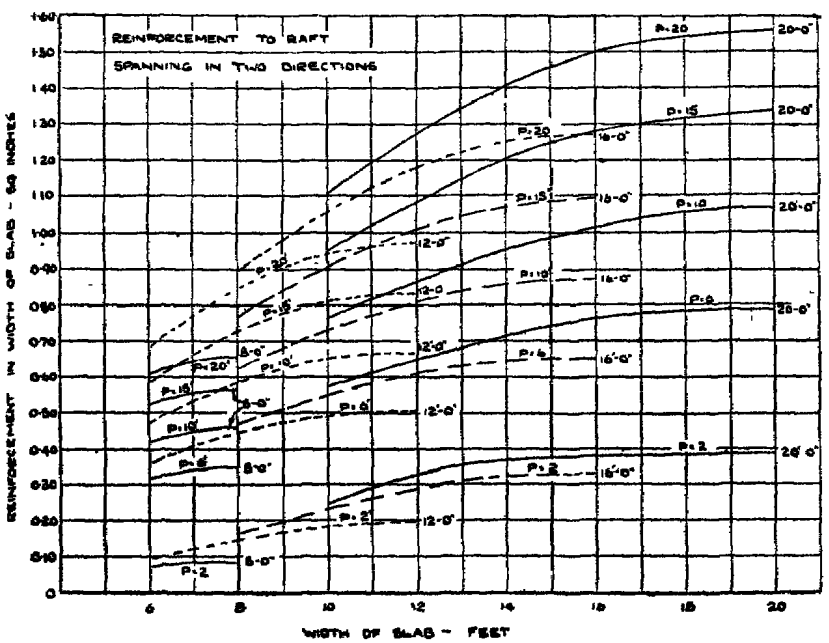


Fig. 13. Steel reinforcement required in rafts.

Rafts

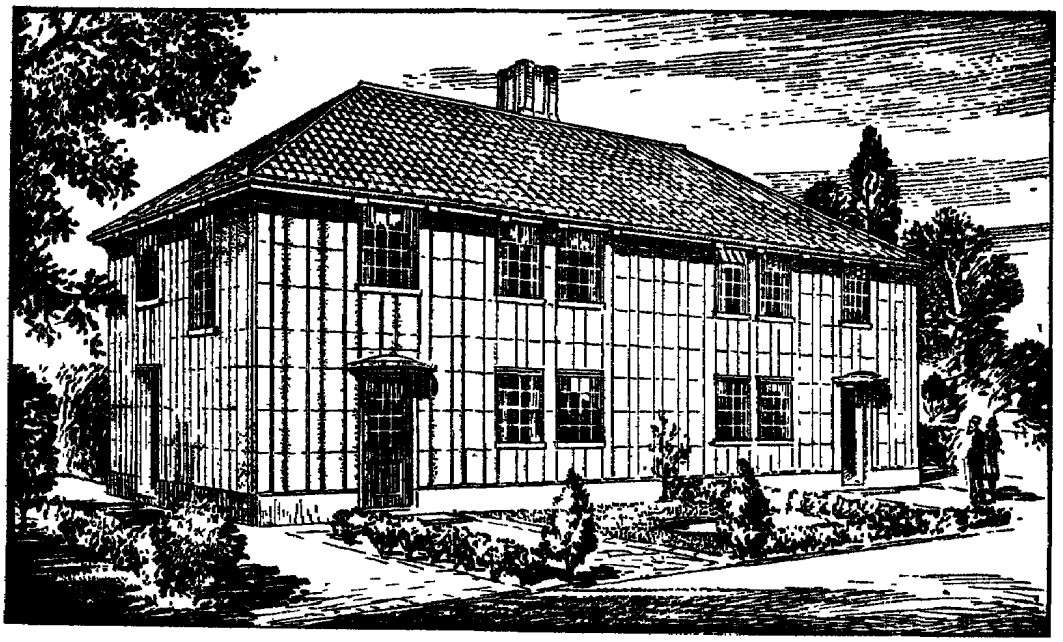
These are always necessary for subsurface structures or those at ground level The design is dependent on the structure and the bearing pressure and hydrostatic head of water Rafts of plain concrete are thick and uneconomical For new work the raft can be monolithic with the walls or independent of the latter, which may consist of brick or masonry The raft is designed and constructed to resist ground and hydrostatic pressure which tend to produce bursting of the floor Reinforcement is necessarily mainly in the top of the raft, and if the basement is a monolithic structure relief of the top raft tension is obtained from the rigidity of the walls It is advantageous to form a display between the footing and raft to provide gradual variation in stress from the footing to the raft. (Fig 10) It will be essential to provide a rendering to both the top of the raft and the external surface of the brickwork.

For cases of brickwork walls on existing floors which are not considered structurally adequate, or where the site contains water of variable pressure, it is sometimes necessary to provide an extra slab on top of the existing floor. The former should be designed to fully withstand all possible hydrostatic pressure, which will be imposed on the new slab due to percolation or water ensuing from cracks in the old slab.

It is generally sufficient to provide 3 or 4 in of wall bearing to the new slab

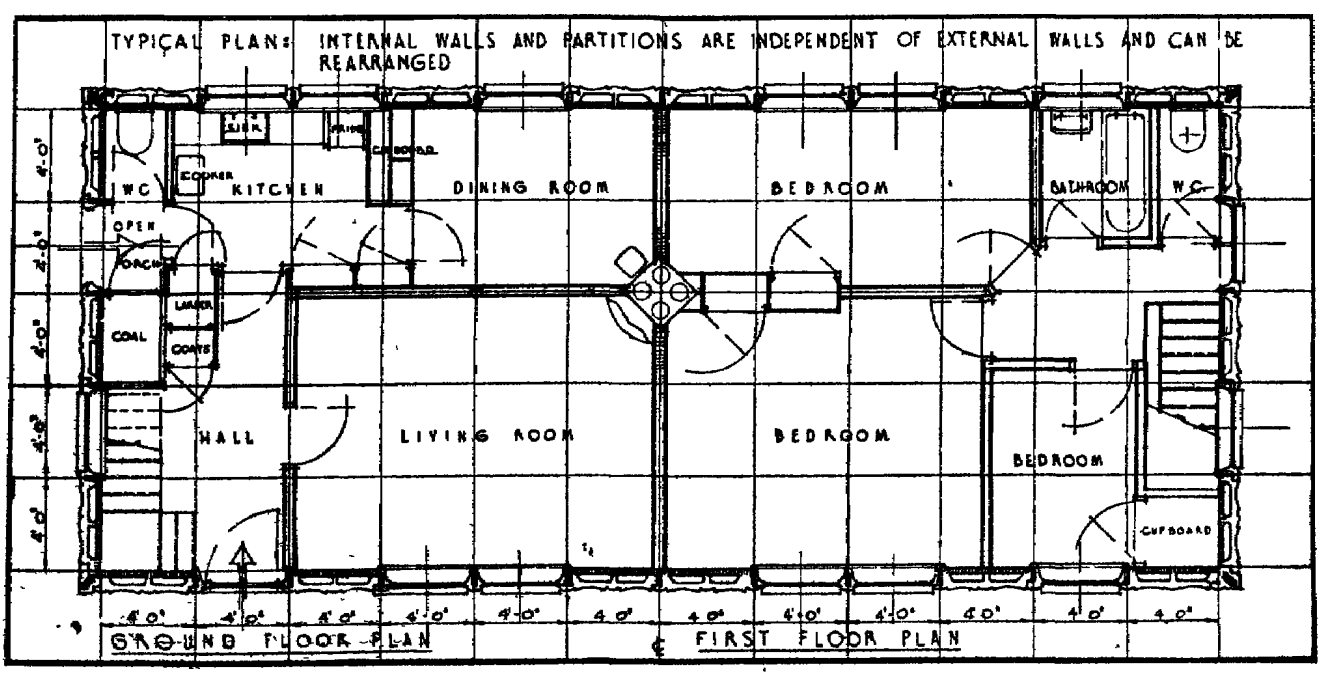
Figs. 11, 12 and 13 give thicknesses of raft and reinforcement required for different water pressures In such cases, it is preferable to anchor the slab all round by cutting the chase in the complete perimeter of the bearing walls. This facilitates impermeability in addition The stresses used are 18,000 and 750 for mild steel in tension and concrete in compression respectively. Since bearing is developed all round the slab the reinforcement is considered as spanning in two directions This is a more economical

(Continued on page 258)



A HOUSE WITH FLUTED WALLS

CHIEF feature of the Mac-Girling system of house construction, of which an example is here illustrated, is the use of walling slabs of Girlingstone This material, which is made in various colours, has been used for many years in public buildings, churches and bridges A form of simple construction has been evolved for house building, of which the basis is a hollow concrete block 4 ft wide by 2 ft high by 11 in deep These blocks interlock so as to produce a semi-framed structure Externally the blocks are fluted vertically, a treatment which gives a play of light and shade and relieves the monotony of large interlocking units The surface is bush hammered. Internally the blocks are formed of cellular concrete and can be finished with a skim coat of plaster Bonding is by means of a 9 in external and 2 in internal horizontal lap The same shaped block is used the reverse way up for alternate courses The joints are arranged so as to incorporate continuous vertical steel reinforcement, thus forming structural weight-carrying piers every 8 ft, and allowance is made for slight expansion in the centre of each Continuous reinforcement, incorporated in the eaves blocks, is linked to that of these piers so as to form one framed structure The architect of the pair of houses in the illustration is Mr John E M Macgregor, F R I B A —(*With acknowledgments to "Illustrated Carpenter and Builder"*)



CONCRETE ROADS

The Holloway-Radial Scheme for Concrete Carriageway Intersections.

By **S. A. HOLLOWAY, A.M.Inst.H.E.**

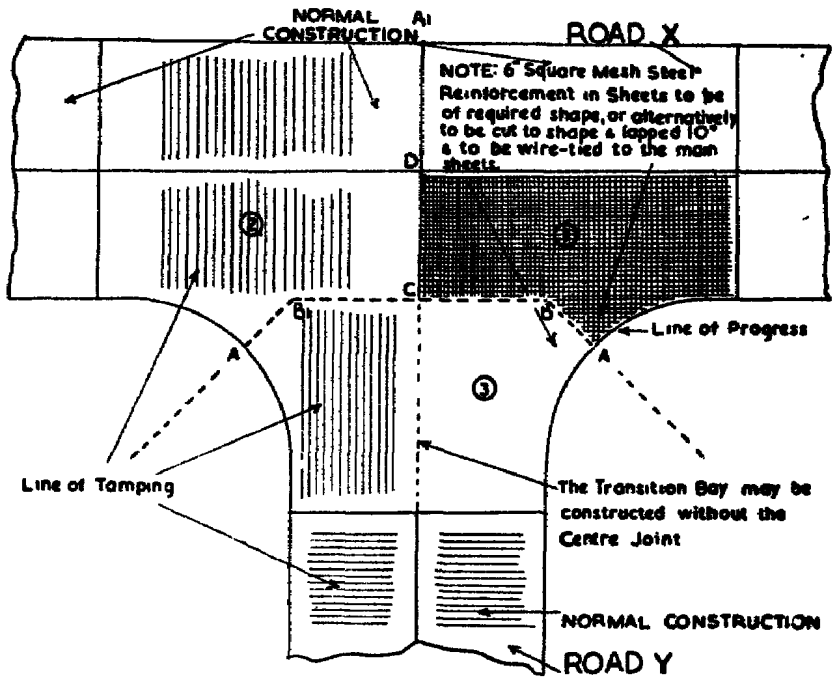
THE object of this scheme is to simplify the construction at the intersections of concrete carriageways The two main advantages are that there are no angles of the concrete which are less than 90 deg and that the construction can be carried forward with the minimum of delay and trouble

The radial form as shown on the plan is the continuation of the line from the centre of the radius of the circular kerb to the point where the right-angle of the kerb lines of roads " X " and " Y " is found

The parallel form is the line of the channel of road " X " produced at point " B " and continued through point " C " to point " B1," where the other radial form is fixed

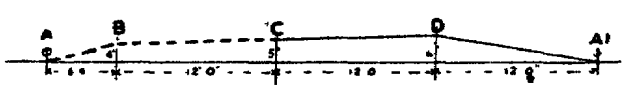
On the plan the radial forms are 6 ft 4 in in length, which is determined by the length of the radius of the circular kerb, which in this case is 15 ft A minimum length of 6 ft is desirable for the radial form The length of the parallel form is 24 ft —*i e*, the width of road " Y "

The plan is based upon that of a site which is flat, and the centre of the scheme



PLAN

SCALE 1/10 INCH TO ONE FOOT



SECTION THROUGH A-A1

SCALE - Horizontal 1/10 Inch to One Foot

" Vertical 1/2 Inch to One Foot.

on road " X " at point " D " has had to be raised 2 in. above the normal rise of the crown, making 6 in at that point, and " boned " from either direction of road " X " and of road " Y " at point 90 ft back If there is a longitudinal fall on road " X," this rise may be minimised or dispensed with altogether. The falls to the gullies where situated have been fixed and are in addition to the above, which is based upon a 5 in normal kerb face

The vertical section through A—A1 gives the general rise of levels of this site. The end of the radial form at point " B " is raised 4 in above the level at the channel at point " A " The level of the paralleled form at point " C " is raised 1 in above point " B " and the level at point " D " is raised a further inch above point " B "

All the forms are to be sited and fixed in advance of the work, and the construction will be carried on with the normal work, of which the concrete sections Nos 1, 2 and 3 are exceptions

When section No 1 is to be constructed the 12 ft. tamper of the normal work will be used for the first 6 ft of progress, and from this point to point " B " another tamper of 17 ft in length The first 12 ft. of this tamper from the channel being shaped as for the normal tamper, the remaining 5 ft to be of a flat trajectory from the 12 ft to the 17 ft length This tamper will cover the area and line of progress When this tamper has reached the maximum at point " A " the crown end will coincide with the centre line form, and this position for progress should be continued allowing the channel end of the tamper to overlap into the transition bay area On reaching point " B " the camber will have become nearly flat, and another 12 ft tamper will be used which will be nearly flat, having a maximum rise of ¼ in at the centre from the chord of the arc, this being necessary to offset the tendency of a concaved surface. This will be continued to point " C " and will have completed the concrete section No 1

The section No 2 will be constructed in the reverse order of operations The normal concrete sections of road " Y " having been constructed, it remains for No 3, the transition bay of 20 ft length, to complete the scheme. This is done by tamping longitudinally, after the edges of all the sections concerned have matured sufficiently The centre expansion joint as produced from road " Y " may be omitted if desired The fine lines show the line of tamping for the various sections This scheme can be adapted to any other type of concrete carriageway intersections.—(*With acknowledgments to " Contract Journal "*)

(*Continued from page 256*)

design than one-way reinforcement The slab is assumed to be simply supported on all sides Thus, all the reinforcement is placed in the top of the slab To completely waterproof the basement in this case, it is necessary to lay a rendering, before the slab is completely set. (Fig. 7)

Facings

Modern reinforced concrete construction has not improved in appearance Precautions with regard to finishing which were taken fifteen to twenty years ago have not been applied as rigidly as they ought Keen competition has ruled out these very necessary aids to a more durable finish, and the need for economy in view of slump trade conditions has not improved this state. Modern methods are often very slap-dash. The use of a richer facing mix is an expedient that fully justifies the extra cost (Fig. 9.) The facing mix should be of the plasticity of putty After the movable shutter is raised, the coarser mix works forward but does not smother the facing mix Another excellent finishing is to take back the heavy aggregate from the mould face, thus allowing the matrix to work to the front Vertical joints show a good appearance if cut at the surface at the correct distance and slightly bush hammering of the face produced. By this means the shutter board marks are completely deleted.—(*With acknowledgments to " Civil Engineering."*)

THE INDIAN ROADS AND TRANSPORT DEVELOPMENT ASSOCIATION LTD.

Speech of the President, Mr. T. R. S. Kynnersley, O.B.E., M.C., at the annual meeting of the Association on Thursday, the 31st October 1946.

WE have just completed a year of that post-war period, for which we had been planning and preparing, and while the year has not fulfilled all the expectations formerly entertained, a number of plans for rebuilding and reconstruction have been finalised or are on the way to being implemented.

There are plans to develop the economic welfare of the country through stepping up food production, constructing irrigation works, opening new industries, &c There are plans also to promote social welfare through extending educational facilities and improving Public Health. Side by side with these, there are also plans for road construction.

The target of the road plan is the programme recommended by the Chief Engineers involving the improvement of the existing 300,000 miles of road and raising it to a total of 400,000 miles during the next 15 years Considering the area of the country, which is one million eight hundred thousand sq. miles, the increase proposed over the next 15 years is just one mile of road for an area of 18 sq. miles ! Or if we consider the number of villages in the country, namely 700,000, the road programme, on an average amounts to no more than one mile of new road for 7 villages in 15 years.

That this programme is far short of actual requirements was demonstrated by the investigations conducted by the Provincial Governments individually in 1944 and 1945, when many of them found that the provision of reasonable road access to all important villages would involve doubling the programme Lack of finance they say, however, will not permit them to contemplate any such increase

It must of course be obvious to the most casual observer that our 700,000 villages—even if they are to have on an average no more than one mile of road each to contact with neighbouring villages, markets and railway stations—require a total road mileage of 700,000 miles, as against the existing mileage of 300,000. It is not surprising therefore that in the ratio of road mileage to the country's area, India's road system, representing one-fifth of a mile of road to the sq mile, is barely one-tenth of that of a country like England which has 2 miles of road to the sq. mile.

What the lack of an adequate road system means can be very easily understood when it is recognised that neither economic development nor the development of social services can proceed satisfactorily without roads. If road development had kept pace with developments in other spheres, that is in proportion to the increase in population, agriculture, industries and commerce during the last 50 years, the country would now have been enjoying a much higher per capita income In addition the amenities of civilization in the cultural, educational, hygienic and social spheres would have been far more advanced

As we have nearly 90% of our population living in the villages, any effort to improve the lot of the common man must begin at the villager's front gate Roads are necessary to bring large tracts of inaccessible and untilled lands under the plough to facilitate the distribution of good seeds and fertilisers to the farmers to cheapen costs of agriculture and thus induce the farmer to produce a surplus which he can take economically to the market, they are also necessary to prevent perishable produce from running to waste, to link industry with people and develop new materials and industries. The same is the case with social services ; and pointed attention was drawn to this by Sir Joseph Bhores' Committee in their recent report on Public Health. " Unless village communications are developed ", the Committee said " the whole plan for rural areas may either be paralysed or lose much of its effectiveness ".

One country that has achieved great results by planned development, in recent years, is Russia, where conditions 25 years ago closely resembled those of India What is the secret of Russia's phenomenal achievements in these 25 years ? Within 5 years of the commencement of planned economic development, I am told, illiteracy was on the decline and village libraries, constantly supplied with new books, sprang up everywhere, bringing the precepts of a new civilisation Hospitals were established , the harvest of the land increased in size by new methods of agriculture , and famines, common to many secluded areas in winter, diminished in size and intensity Russia began to grow prosperous

But long before the educational and other groups were able to traverse the countryside, Lenin was building thousands of miles of roads, even to the remotest villages, using for their construction legions of unemployed

If it had not been for these roads, none of these results could have been achieved For several years the Soviet Union constructed roads at the rate of 30,000 miles per year In this lies a lesson for this country which we cannot afford to ignore

It is to be hoped therefore that the Central and Provincial Governments will do everything possible to promote the speediest execution of the road plan and when the present target is achieved, endeavour to extend the mileage further. At present only one third of the road plan, to cover the next 5 years, appears to have been approved, at a cost of roughly Rs 150 crores It is satisfactory to note that some of the Provinces such as Bengal, U P , Assam and Sind have commenced the programme and have sanctioned substantial grants in the current year's budgets Other provinces—and particularly Bombay and Madras—have not yet given final sanction for the commencement of schemes on any substantial scale It is hoped that the Provincial Ministries will see to it that no avoidable delay occurs

Coming to road transport services, there are ominous reports that certain Provincial Governments propose to nationalise road transport It is surprising that Government should even contemplate such a step when the examples of Germany, Northern Ireland and such other countries which experimented with nationalisation point to the folly of attempting such a step

The road transport industry in this country is comparatively young and still in an expanding stage, where incentive to explore every possible method for the improvement of services is the first essential Is such improvement likely to be achieved more rapidly or more effectively under state ownership than under private enterprise ? The answer seems obvious enough Private enterprise alone can ensure full scope for the development of road transport and ensure that the public will enjoy the full advantage which road transport is capable of providing

Flexibility is an important characteristic of road transport and this will be lost under an organisation where control is highly centralised Personal service is equally important The road transport operator is in intimate touch with the conditions prevailing in his locality He is therefore in a far better position

to anticipate demand and supply than an over-centralised organisation can ever be

It is not for a moment suggested that the Government's attitude towards this industry should be one of 'laissez-faire'. There is a middle course between 'laissez-faire' and nationalisation and that is to provide adequate safeguards and to give force and direction to the industry's orderly development

There have been problems and defects—and annoying ones at that—in the past but if we look back upon the history of road transport development, we find that one by one they have been solved Fitness and road worthiness of vehicles have been ensured by the Motor Vehicles Act Unregulated and wasteful competition has been curbed by the licensing system The latest step in this direction is 'compulsory third party Insurance' This meets the long felt need to provide cover against risk to which human life is exposed, this risk has been increasing as growing mechanisation brings more and more complications in almost every aspect of human existence

In a growing industry like that of road transport, progress can only be step by step and it has to be within a framework of healthy competition, allowing initiative and enterprise, skill and resourcefulness, full scope, so that the community may enjoy the very best that road transport can give But if nationalisation is introduced and it proves a failure, retraction would be impossible and the consequences disastrous The best interests of internal transport would be served surely by an approach to the problem of co-ordination of all forms of transport on the basis of the policies followed in England and America.

Another important matter to which I should like to draw your attention is the proposal of the Government of India to restrict road transport in this country The proposed code seeks to impose a large measure of restriction on the operation of road transport on routes exceeding 50 miles in length and curtail very severely goods transport on routes of over 100 miles

Generally speaking it is true that road transport should be for short and railways for long distances but a normal and effective haul for a motor lorry can be taken as a day's run which may be considered as being at least 250 miles on good roads, and it seems to me to be the height of folly to try and curtail road transport over such a distance.

My Council and I deprecate most strongly any spirit of hostility between roads and railways. They are both out to serve the great Indian public, the sole aim should be efficiency.

Healthy competition is always good and makes for efficiency and the progressive railway-man will not contradict this.

Other countries have not found that competition between road and rail is harmful to one another. On the contrary, competition has proved to be a stimulant to the provision of better services, greater efficiency and greater comfort and convenience to the user and all these are very badly needed in this country Today, a year after Japan's surrender, India is still suffering from inadequate transport facilities The Railways cannot carry the freight offered and passengers, whatever the class they travel by, have to put up with all sorts of discomforts and inconveniences An evil that seems to have of late assumed enormous proportions is that of pilferage on the Railways and the commercial community is becoming increasingly alarmed and bitter over this Instances could be cited where whole wagonloads have disappeared altogether—others of urgently required materials not reaching their destination even months after being dispatched, consignments arriving at destinations with contents seriously diminished due to pilferage in transit has become a common occurrence these days

When such conditions prevail, the policy needed is one of encouraging competition and not of introducing measures which will arrest reasonable road haulage and passenger transport in this country The present competition between road and rail here is insufficient to produce any good results

On one side physical restrictions brought about by regulations or lack of roads and on the other excessive taxation of the motor vehicle and its fuel, have combined to deprive the country of the advantages of cheap road transport I should like to say a few words on the question of taxation It is needless to point out that the effect of high taxation on any machine tool would be to put a brake on every economic activity requiring the service of that tool, and as industry has no more valuable tool than the transport vehicle, national production cannot but suffer from excessive taxation of transport The time has therefore come not only to cancel the war time taxes on motor transport, but also to reduce the incidence of the pre-war taxation Of the war-time tax of 5 annas on petrol, 2 annas still remain which I suggest should be done away with as the very first step.

Roads are making an extremely heavy contribution to the general revenues of the country. At the outbreak of the war, motor transport in British India was paying roughly Rs 10 crores per annum in taxation to the Central and Provincial Governments, as compared with an expenditure of no more than 6 crores on road construction and maintenance But for petrol rationing, this revenue would have steadily increased. From the standpoint of the exchequer, excessive taxation is bound sooner or later, to bring into play the law of diminishing returns in various directions. By crippling motor transport, it tends to restrict the expansion, if not to create a diminution of, provincial revenues which depend on the prosperity of commerce, industry and agriculture. Overburdening motor transport with taxation at prohibitive rates is therefore nothing short of killing the goose that lays the golden eggs.

Lastly I should like to refer to the Motor Vehicles Amending Bill Quite a comprehensive Bill to amend the Motor Vehicles Act of 1939 was introduced in the Central Legislative Assembly in April and is to be examined by a Select Committee of that house It seeks to remedy certain minor defects disclosed in the operation of the Original Act and, under the name of co-ordination, to employ the Motor Vehicles Act as the machinery for eliminating road competition with Railways. One can quite understand how co-ordination can be secured by an authority if it has power over both roads and railways But if you have an authority with power only over road transport and if it is charged with the duty of seeing that railway interests are safeguarded, it cannot result in co-ordination but only suppression of the road in favour of the rail This apparently is what the Amending Bill seeks to do It proposes to empower railways and providers of other forms of transport to object to the issue of road permits and makes it obligatory for the Transport Authorities to enforce the policies agreed to between the Central Government and the Provinces Harmless, though the latter provision is at first sight, it may be that position is that Provinces are being coerced by the Centre into agreeing to restrict road transport on pain of losing the promised financial assistance in the construction and maintenance of National Highways The Central Government appropriates to its general revenues more than 60% of the total revenue contributed by motor transport but it looks as though its proposal to take over responsibility for the relatively small mileage of National Highways is dependent on Provinces agreeing to restrict road transport, not only on those National Highways, but on the entire road system of the country The proposals of the Centre which are sought to be implemented though the Motor Vehicles Amending Bill seem definitely unfair and I sincerely hope the Central Legislature will remove the offending clauses from the Bill before adopting it. Otherwise not only will the individual lose his inherent right to transport himself and his goods by the methods most suitable and economical for his purpose, but also a real disservice will be done to the ultimate prosperity of railways themselves For it is clear that a concern which relies on the superiority of its service for attracting custom learns to increase its efficiency from day to day while one which is taught to force its services on the community though the elimination and suppression of alternative modes of transport is bound, like a pampered child, to lose all initiative and enterprise and become a burden on the country

TRIALS OF REINFORCED CONCRETE ARCHES ON ROADWAYS

USEFUL FOR PERMANENT ROADS IN SETTLED GROUND WHERE CORROSION IS A MAJOR PROBLEM

By **W. H. EVANS and A. WRIGHT***

EARLY in 1942, when the future supply of steel supports was precarious, steps were taken by the Lancashire and Cheshire Safety in Mines Research Committee to investigate the possibility of using reinforced concrete arches in place of steel arches for the support of roadways Face supports, in which all possible economies had been pursued hitherto, actually account for but one-tenth the total consumption of steel for supports, whereas a total of 250,000 tons per annum is used in the form of arches and girders for roadways, hence a fractional reduction in steel used in roadways would outweigh the total steel used at the face This point was fully realised and several attempts were made during the war to introduce concrete linings on roadways, including concrete arches

The object of the present investigation was to provide a reinforced concrete support equal in strength to steel arches, having regard to the requirements for transport and erection underground It should be emphasised that it was not the intention to compete in design with those systems of concrete lining used extensively in Germany, which usually form a continuous lining, often with an invert, and are built at considerable expense The object was merely to seek a ready substitute for steel arches in the event of further wartime restrictions on supplies Fortunately, this did not occur, but the outcome of the tests is of considerable general interest

Design and Manufacture

An approach to the Cement and Concrete Association met with a ready response and offer to co-operate in the investigation by supplying arches for trial The Association appointed Dr Hajnal-Konyi to act as their consulting engineer, and he produced the final design after discussion of the requirements of strength, flexibility, and facilities for erection Full particulars of the design for a 9-ft by 12-ft arch are shown in Fig. 1.

The arch was of nearly parabolic form, corresponding to the splayed-leg type of steel arch, and comprised five members coupled together at the ends by flexible and yielding joints Each member was about 5 ft. long and weighed about 170 lbs (1½ cwts), so that two men could handle it. The cross-section of concrete was rectangular, 8 in. deep by 4 in wide, and the reinforcement consisted of four rods ½ in. dia, with stirrups set at 3-in centres and at 1-in. centres near the ends. It will be noticed that the reinforcement was placed well to the outside of the section, with only ½ in of cover over the main rods, thus utilising the steel to the best advantage A sheet-steel collar was placed at each end as additional reinforcement and to carry a folded steel strip which retained softwood crushing pads 2 in thick To prevent dislocation of the mating ends by shear forces, a key plate ¼ in thick by 5½ in deep was passed through and at right angles to the line of the joint, as shown in Fig 1, and retained by a bolt At the feet, semi-circular timber pads were secured to the end collars by a strap Thus the three members forming the crown of the arch were identical and interchangeable, and the two lower members, which were struck to a larger radius, were easily identified for position

The design included reinforced concrete struts to preserve the alignment and proper spacing of the arches at 3-ft centres Ten struts were used in each panel, *i e*, two to each member, set near line The struts were 8 in by 4 in in the ends of the members and in continuous overall section, but were cored out for lightness and to enable the fastening bolts to be inserted at the ends, one bolt ¾ in dia served for two adjacent strut ends. Each strut weighed about 56 lbs.

Reinforced Concrete Arches on Roadways

To assist the design of the arches tests were made to decide the mix of concrete to be used, and on individual arch members and struts to prove their strength The concrete used in the arches was made with ordinary Portland cement, sand, and crushed granite passing 3/16-in sieve, in the proportion 1 1 2 and the average strength of 6-in cubes was 4,450 lbs per sq in at 15 days, this indicated a strength of 5,500 to 6,000 lbs per sq. in at 28 days

Test of Curved Members

Three curved members were tested separately when set between rigid skew-backs with load applied equally at the quarter span points through hardwood blocks Each member was 4 ft 9 in long measured along the chord at mid-depth of section at the ends The results of tests are given in Table I

In the course of each test the timber end-pads were compressed and, in specifications Nos 1 and 2, the ends of the member subsided on to the base of the machine At maximum load the timber pads were reduced to about 1-in. thickness and the subsidence of the crown was 4 8 in on average First cracks occurred at loads between 9 and 15 tons The residual load carried after failure was about 10 tons, or nearly half the maximum load It was decided to adopt the construction used for Specification No 3 in the arches for trial underground, and they were made by Hydroprest Concrete, Limited Altogether twelve struts were tested, some in compression and some in tension Under uniform compression the average strength was 26 tons, and with load applied over a 2-in wide strip adjacent to one short edge, the average strength was 12 tons The average tensile strength was increased from 4 tons to 7 tons by improving the disposition of reinforcement at the ends.

Relevant Points in Comparing Concrete with Steel Arches

In comparing the strength of the reinforced concrete arch (deduced from the results of tests on single members) with that of ordinary steel arches, the fundamental difference in construction and

TABLE I —*Tests on Reinforced Concrete Arches.*

Spec. No	Age	Reinforcement	Maximum load	Type and position of failure
1	7 weeks	4 rods ½ in dia stirrups ⅜ in. dia. welded on at 3 in centres.	23 tons	In shear starting at one load point.
2	12 weeks	As for Spec. No. 1 with extra rod in the intrados and more stirrups at the ends.	31 2 tons	In compression at extrados between load points
3	6 weeks	4 rods ½ in. dia. hooked stirrups ⅜ in dia. at 3-in centres, and ¼ in dia 1 in. centres at ends	22 tons	In shear under one load point (stirrups displaced in construction).

* Lancashire and Cheshire Safety in Mines Research Committee.

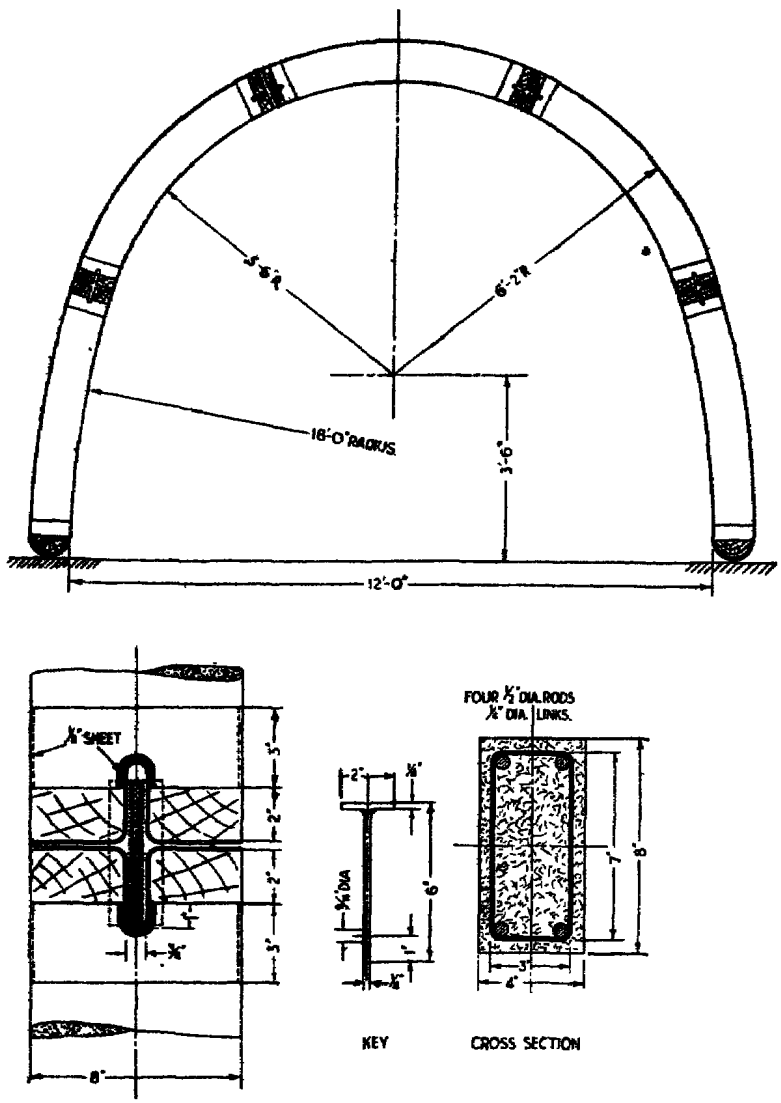


Fig. 1. Reinforced Concrete Arch as Used in Trials.

action must be borne in mind. The concrete arch was articulated at four positions, not merely to facilitate erection, but to reduce the maximum bending moments incurred, and consequently the amount of reinforcement needed. Resistance to strata pressure was obtained chiefly from the compressive strength of the concrete, as is the object in any arched structure, and to that end the section was made deep in the plane of the arch. The more uniform the pressure on the arch, the more nearly is the ideal of uniform compressive stress in the concrete approached To cope with the irregular pressures usually met in mine roadways, the joints were designed to allow a limited sliding movement and substantial angular movement, and then to lock and provide the adjacent curved members with a mutual abutment from which they derived additional strength. The steel collars at the ends of the members helped sustain the concrete under the intense local loading developed in such circumstances. When subjected to uniform compression, the double thickness of timber pads would offer increasing resistance while allowing nearly 3 in of yield or shortening in the periphery of the arch Because the pads were distributed around the arch, they were more effective than a similar total thickness as used in foot blocks for steel arches The method of controlling displacement caused by shear forces by means of key plates set across the joints was ingenious and effective, although the results of the trials showed that the plates could be made stiffer with advantage.

In steel arches resistance is developed in part by true arch action, but usually to a greater extent by the bending strength of the section The high ductility of steel permits large deflections without recourse to special hinged joints; the joint at the crown is provided merely for easy transport and erection, and is often made nearly as strong as the full arch section.

Comparative Strengths of Straight Members

The maximum strength of a straight piece of steel arch section of the same length as the concrete members and similarly loaded has been deduced from tests on specimens 4 ft. 6 in. long tested in simple bending. The section of steel arch used in roadways adjacent to the concrete arches on trial was 5 in. by 4½ in and the maximum load for specimens of this type was computed at 35 tons. For 4-in. by 2½-in equal flange section of medium-carbon steel the load was 28 tons, and for the same section in mild steel, 18 tons A 4-in. by 4-in. section in mild steel would carry 18·5 tons The deflection at maximum load would be between 6 and 10 in and the strength would fall fairly rapidly as the section deformed.

Under transverse loading, therefore, the reinforced concrete members as used in the trials were about equal in strength to mild-steel arch sections weighing 11 to 13 lbs per ft run, *i e*, of 4-in by 2½-in or 4-in by 4-in section The weight of steel reinforcement was 5 5 lbs per ft run, or about half the weight of the steel section of equal strength Naturally the strength of the concrete would increase with age and thus increase the strength of the arch On the other hand, steel arches would gain considerable strength by arching, if buckling is prevented by good struts between the arches

Wartime Reinforced Concrete Road Linings

Reference may be made here to other designs for reinforced concrete road linings advanced during the war One support* comprised five members, as in the present design, but they were straight and had mitred ends, the only provision for yield or distortion being thin wooden wedges inserted at the tops of the leg members to give a splay of 3 in on each side

Another type of concrete arch was designed and produced by the North Notts Gravel Company, Limited, for use on a return airway slant 6 ft. wide by 6 ft. 6 in high. It was made like a steel arch, in two parts joined at the crown by steel fishplates The section of concrete was 5 in by 5 in, with heavy flutes in the sides to make an H-section with web 2½ in. thick Four rods of ⅜ in dia were set in the flanges and held by stirrups ⅛ in dia. at 6-in centres. The arches were of approximately parabolic shape, straight in the leg for 2 ft 3 in from floor level and then curved at 5-ft radius and (near the crown) about 1-ft 6-in radius There was no provision for yield or distortion, because the arches were set in settled ground and under a mere 270 ft. of cover Forty arches were supplied for this trial installation, for which the incentive was to use a material more resistant than

* "The Uses of Concrete Underground, with special reference to Concrete Props" L. C. Maitland, Trans.I.Min.E., Vol. CII, 1942-43. See discussion by Mr. McPherson.

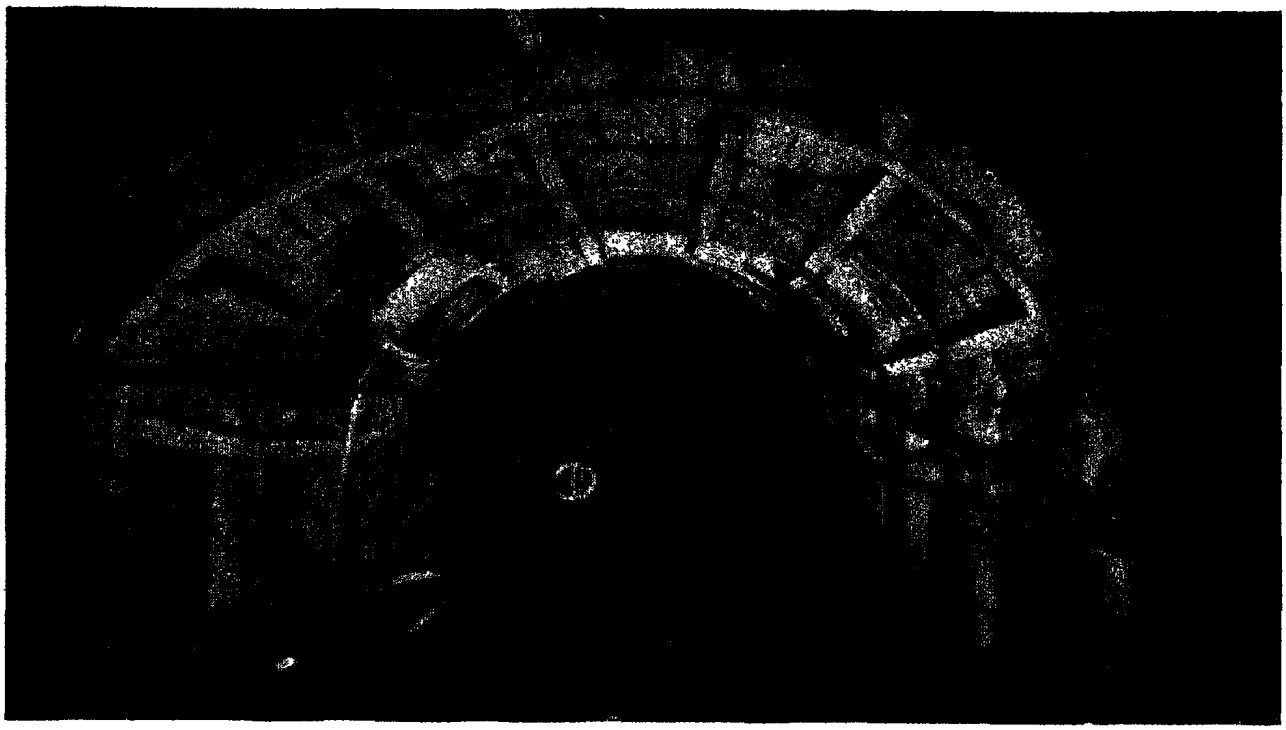


Fig. 2. First Installation of Four Arches Showing Condition after Three Years.

timber to moist conditions They were standing well twelve months after erection

A set of arches was manufactured by the Coalmoor Basalt Company, to the design of one of the writers, but they were not erected underground This arch was for a 12-ft by 9-ft road and was of parabolic shape It comprised four members, the lower ones weighing 224 lbs each and the upper ones 180 lbs The section of concrete was 8 in. by 4 in and it was reinforced with four rods and stirrups Hinges were formed at the three joints by making the adjacent ends of the members concave and inserting a short length of round timber, 7 in. dia. The bottom ends of the lower members were tapered to allow them to penetrate the floor A simple form of strut, consisting of a single $\frac{3}{4}$-in rod screwed at both ends and encased in concrete of 4-in square section, was used, with staggered holes in the arch members

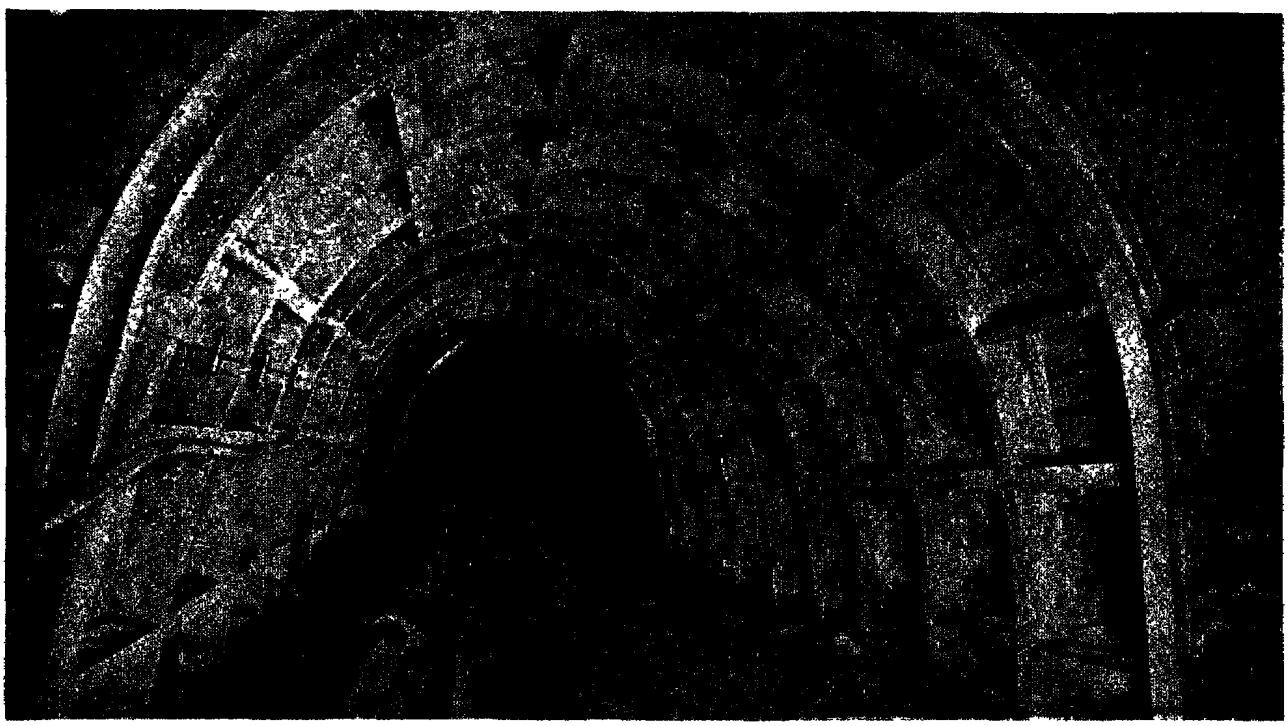


Fig. 3. Second Installation as Set, Looking Down Dip.

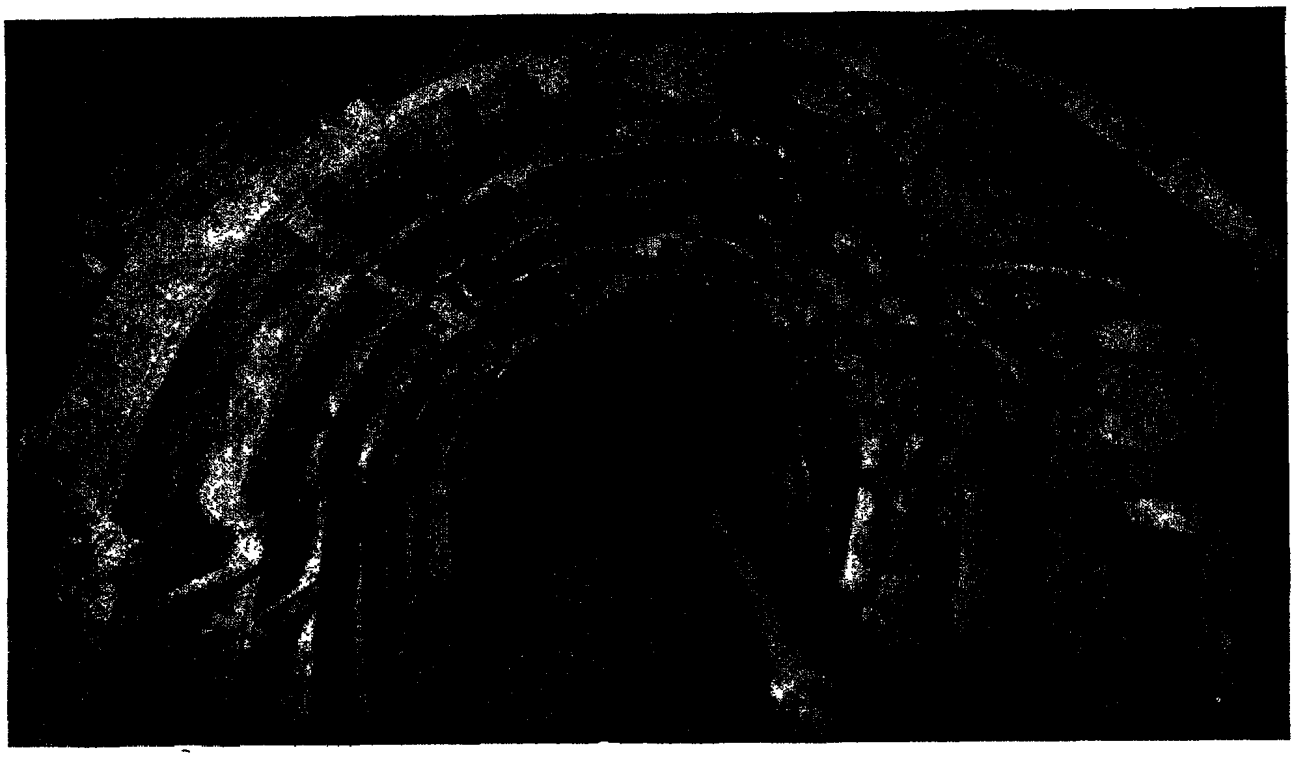


Fig. 4. Second Installation 2½ Years after Setting, Looking Up Dip.

The object of this design was to achieve the maximum simplicity in construction and erection Tests showed that the joints allowed 3-in yield under direction compression, no special provision for shear was made at the joints, because the arches were intended for trial on a road in soft ground subject to heavy pressure all round

Underground Trials

The underground trials were made at a Lancashire colliery where particular interest was shown in the investigation The first trial was with four arches only, set in relatively settled ground near the edge of the shaft pillar in a tunnel rising 1 in 15, the depth of cover was about 600 yds Previously the roadway had been lined with 4-in by 2-in by 2½-in section steel arches which had suffered considerable deformation The concrete arches were put in adjacent to 5-in by 4½-in section steel arches when the roadway was re-ripped No special facilities were available for setting the arches, which was carried out by four men It was estimated that the time required was twice that taken by three men to set similar 5-in by 4½-in steel arches This drawback could be reduced by using a wooden template to facilitate positioning the individual concrete members.

After 5½ months there was some evidence of crushing and distortion of the timber-filled foot-pieces and one strut showed signs of cracking There was slight reduction in height and width and a small displacement at one joint Continued slow ground movement was accommodated by further crushing of the timber pads between the joints and at the foot-pieces, several joints were displaced, but securely retained by the key plates Many of the struts developed cracks without failing, a feature which will be mentioned later A general view of the installation after three years is shown in Fig 2, when the average height was 7 ft to 8 in and width 11 ft 6 in

In order to provide a more severe test, ten more arches were set in a roadway 50 yds from and parallel to a new face, which started working from the development three months after the arches were set, the depth was 660 yds The roadway, which was elsewhere supported by 5-in by 4½-in section steel arches, was on a dip of about 1 in 7, and the site of the arches was about 170 yds behind the developing face In the vicinity of the arches the roof and sides were formed of a strong sandstone 9 ft to 12 ft thick overlain by a blue shale The roadway was flanked by 10-yd packs, followed by 8-yd wastes and 4-yd waste packs The general appearance of the arches after setting is shown in Fig 3 Owing to a shortage of concrete struts, a number of round timber struts were used instead, chiefly at the feet of the arches, two struts were omitted where a man-hole was required on the right of the road

After ten months, during which time the face had advanced 160 yds the arches were showing definite signs of load, although they continued to give adequate support Floor lift was very marked, amounting in the middle of the road to about 3 ft and was largely responsible for the height loss. About this time a floor ripping was made, after which the average dimensions of the arches were: height, 8 ft 3 in, width 3 ft. from the floor, 10 ft. 3 in Almost all the footpieces and joint pads had been crushed to some degree as provided in the design Distortion of the profile by most of the arches had occurred of relative movement of the several members Thirty struts were cracked, eight having partially failed and two completely fractured In two arch members the concrete had spalled and exposed some of the reinforcement, due to pressure from the adjacent pairs of struts

To date the arches have been set about 2½ years and the face adjacent to them finished working six months ago, having travelled some 600 yds Further deterioration of the arches has occurred during that time, but they still continue to maintain adequate support with a good cross-sectional area Their general appearance at the present time is shown in Fig 4 The average section is about 7 ft 8 in high by 10 ft 1 in wide at the floor and 9 ft 9 in at 3 ft above the floor All but 27 struts have cracked and five have failed completely, by transverse loading for which they were not designed. At almost every joint, the softwood pads have been crushed to half their thickness and many of the joints are displaced to the full extent allowed by the key plates These features are illustrated in Figs. 5 and 6 Despite these substantial changes in shape, of the arches there has been no failure in any curved member. Thus one purpose of the design was achieved

In order to afford a comparison, 5-in. 4½-in section steel arches adjacent to the concrete arches were kept under observation. During the 2½ years under review the average width 3 ft. above the floor decreased from 11 ft 6¾ in to 10 ft. 7 in, and the height from 9 ft 2¼ in. to 8 ft.



Fig. 5. Showing Reinforced Concrete Struts Deflected and Cracked and Differential Sliding and Angular Movement of Curved Members.



Fig. 6. Close-up View of Joint showing Key Plate Controlling the Sliding Movement

2 in The steel arches have suffered only slight distortion and are in good condition

Results of Trials

Several useful results were obtained from the trials Firstly, as to the original object, it was shown that concrete arched supports could have been used as a suitable substitute for steel arches under certain conditions should a severe shortage have occurred Their use would, however, probably have entailed more labour in erection and required more ground to be excavated to receive them The method of joining the members proved satisfactory, although open to minor improvement, it allowed considerable angular and linear movement between the members, thus giving flexibility to the support to accommodate uneven pressures All damage sustained was in the struts or attributable to their failure, and this could probably be avoided by using struts which do not project to the backs of the arches Of the arch members themselves, none was seriously damaged; and therefore they could be used again after replacing the timber crushing pads, which is a simple matter This is important, because it shows that a serious criticism of reinforced concrete props, that they were not reclaimable, does not necessarily apply to articulated concrete arches

An incidental advantage of the deep solid section used for the concrete arch was that the illumination and appearance of the road was improved As shown by the photographs, the lagging and ground exposed between the arches was hidden from the forward view, and the reflecting power of the concrete was more satisfactory than that of the adjacent steel arches The continuous lines of struts added to this effect

In conclusion, the trial justified its inception under the conditions holding or threatening at the time Under present conditions of supply, it is unlikely that concrete arches can compete with steel arches for the support of roads in live ground On the other hand, they may come into consideration for permanent roads in settled ground, particularly where corrosion is a major problem -- *(With acknowledgments to "The Iron & Coal Trades Review")*

ERRATA

(Design of Foundations from Stand-Point of Soil Mechanics, printed in the "Indian Concrete Journal," October 1946, on pages 226-230)

Page	*Column*	*Line*	*For*	*Read*
228	1	24	edge B	edge Q
,,	1	31	DBG	SQM
,,	1	43	$h\sqrt{\tan \phi} = c^1$	$h \gamma \tan \phi = c^1$
,,	1	63	$\sqrt{H}$	γH
,,	2	1	$q = H\sqrt{} = Z\sqrt{} K_p^2$	$q = H\gamma = Z\gamma K_p^2$
,,	2	15	$q = b \sqrt{} K_p^2 \left[1+\left(\frac{z}{b}\right)+m\left(\frac{z}{b}\right)^2\right]$	$q = b\gamma K_p^2 \left[1+\left(\frac{z}{b}\right) + m \left(\frac{z}{b}\right)^2\right]$
,,	2	20	$q = b \sqrt{} K_p^2 \left(1 + \frac{z}{b}\right)$ $= \sqrt{} K_p^2 (b + z)$	$q = b\gamma K_p^2 \left(1 + \frac{z}{b}\right)$ $= \gamma K_1^2 (b + z)$

(*N B.*—The notation γ, to be corrected on page 228, as per the errata sheet above, actually denotes the notation printed on page 226, column 1, line 15, for the unit weight of the soil)

A FACTORY CONSTRUCTED WITH PRECAST UNITS

A NEW workshop for making precast concrete wall slabs, erected by Orlit, Ltd., at their works at Colnbrook, is constructed almost entirely in precast reinforced concrete units *Figs* 1 and 2 are internal and external views of the factory under construction The main portion of this single-story building (*Fig.* 1) is 300 ft long by 48 ft wide A continuation at one end is 72 ft wide for a length of about 100 ft and a shorter continuation at the other end is 60 ft wide and incorporates the boiler house which supplies steam to the curing tunnels seen in *Fig* 1

An annex, seen in *Fig* 2 on one side of the centre portion, is 36 ft long by 12 ft wide and is intended to accommodate the concrete mixing plant The new building will be devoted entirely to the manufacture of wall slabs, and is planned for an output of 5,000 slabs each day Steam curing is a feature of the manufacturing process. The slabs will be placed in racks that slowly travel through the tunnels, where they will be subjected to a temperature of 160 deg F for a period of four hours The chain conveyors for transporting the racks are below the concrete ground floor

In the 48-ft wide portion of the building there are two rows of outer columns and a single intermediate row The columns, which are spaced at 18-ft centres longitudinally and 24-ft centres transversely, are standard precast units increased in length and reinforcement to give 14 ft headroom The outer columns are 8 in by 6½ in. with a single cantilever head carrying the main beams of the roof. The inner columns are 9 in by 6½ in., with a double cantilever head The lower ends of the columns are grouted into a recess in the foundations

The main beams are standard precast I sections, 24 ft long by 15 in deep by 6½ in wide, when erected they have binders projecting about 12 in above the top flange The secondary beams, also of precast I sections, are about 18 ft long by 12 in deep by 5 in. wide and are at 6-ft centres on the top flange of the precast main beams The 12-in space above the main beams is filled with in-situ concrete deposited between thin precast vertical slabs forming permanent shuttering, and additional longitudinal reinforcement wired to the projecting binders is placed in this in-situ upward extension of the main beams The flat roof is composed of inverted precast tray units 2½ in deep overall and about 6 ft long A screed of 1¼-in minimum thickness is laid on the units to provide falls and a surface to which a double layer of roofing felt is applied.

Along the centre of the building is a clerestory (*Fig.* 1), the vertical members of which are precast and are secured to the longitudinal beams by in-situ connections.

The walls consist of an inner and outer leaf of slabs measuring 1 ft 4 in. deep by 4 ft long by 2 in thick. The two leaves are spaced 8½ in. apart so that they cover both faces of the columns The external finish is described as faced with stucco with a scraped texture finish The principal windows are 14 ft long by 4 ft high and consist of standard metal windows built into precast concrete frames The latter and the door frames (*Fig* 2) are 12½ in wide so as to be flush with the inner and outer faces of the walls.

Overhead runways and similar equipment are supported on steelwork independently of the roof construction

The curing tunnels are isolated from the building structure Eaves gutters are of box sections and are of precast concrete, the sides and bottoms being about 1¼ in. thick.

In the 72-ft. wide portion, seen on the right of *Fig.* 2, the roof is pitched and is carried on steel roof trusses of 36-ft span spaced at 9-ft centres. The sole-plates of the trusses bear on longitudinal steel beams, which are supported on precast concrete columns spaced at 18-ft centres in two outer rows and one intermediate row.—(*With acknowledgments to "Concrete and Constructional Engineering"*)



Fig. 1.—Interior View.

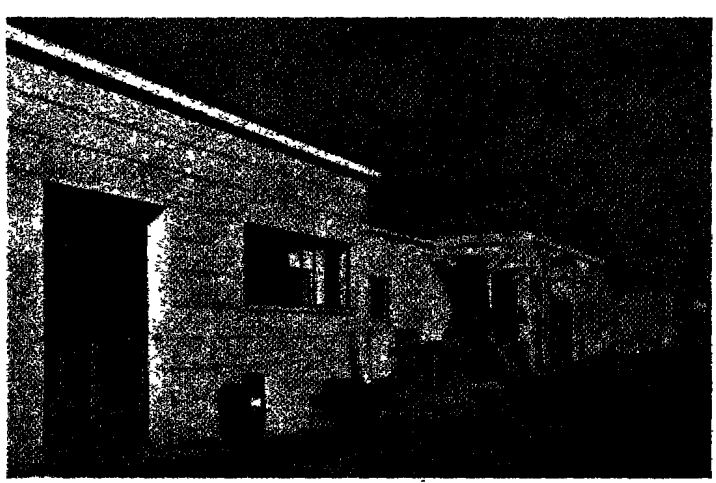


Fig. 2.—Exterior View.



SIMPLE FORMS CUT PILE CASTING COSTS

1. SIMPLE PLYWOOD PANELS, held at bottom by rails nailed to platform and at top by crossrails that also serve as runway supports, make up forms for first stage of concrete pilecasting sequence First forms are so spaced as to permit casting alternate set of piles between those of first set.

INGENIOUS use of one set of pre cast square concrete piles as form supports for an alternate set poured the next day is resulting in unusually low form costs on Tidewater Construction Corp 's pile-casting job for a Navy pier at Norfolk, Va The contractor is taking advantage of the fact that the piles must remain in original casting position on the platform at least seven days for curing Forms for the first group of piles poured on a platform are accurately spaced to permit, after the first forms are stripped, insertion of ¾-in plywood liners against the pile sides as the forms for the second set poured between the individual piles of the first set The liners, plus end bulkheads, are all the forms necessary for the second group of piles.

The piles are 18-in square, reinforced with cages made up complete in advance with welded longitudinal bars, and placed as a unit. Piles vary from 60 to 87 ft in length In the casting sequence, longer piles are always cast in the first group so forms for the second group lie entirely within the slots between the first set

The pile-casting yard is divided into three casting platforms, each accommodating a maximum of 24 piles Only one set of forms, serving all three platforms, is required for the first group, though three sets of liners or intermediate forms are needed for the second group, as they cannot be stripped until the piles are cured and lifted

Along one outer edge of each platform is a permanent form securely anchored and held by outside knee braces This is the anchor for all other panels used for casting the first group The other forms of this set are simple panels made up of plywood sheeting, 2 × 4-in studs and a single 2 × 4-in top rail The panels are spaced and held at the bottom by 2 × 4-in bottom rails nailed to the platform with double-headed nails for easy stripping Notched 2 / 6-in templates placed across the tops of the panels, including the permanent form at the edge, automatically plumb the panels and hold them erect until 2 × 6-in transverse rails can be nailed, also with double-headed nails, across the tops of the side form rails These transverse rails not only brace the form panels, but also serve as the base for buggy runways and working platforms As the side panels are 24-in high, there is plenty of room beneath the transverse rails for finishing Corner chamfers on the piles are formed by ¾-in triangular moulding nailed to the bottom of the side panels and at top strike-off level.



2. PLYWOOD LINERS against piles poured previous day are forms for second group Alternate piles cast in second stage are always shorter than those in first set. Runway supports for second pour are blocked up on first piles.



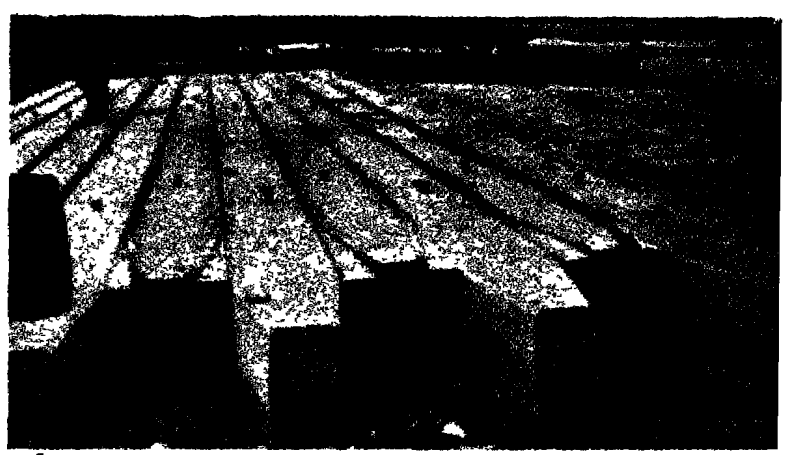
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3. TRANSIT-MIX CONCRETE is delivered to buggies at casting platforms for distribution to forms. Knee-bracing for outside form panel at right of platform is only bracing required for entire form system. Rest of form panels for first group of piles are spaced and held erect by top transverse rails tied into braced panel.

4. LINER PANELS for second group remain in place until all piles are lifted from platform after minimum 7-day curing period. Plugs in top pile surfaces protect lift sockets

5. EYEBOLT LAG SCREWS are screwed into spiral shell cast in the piles for lifting.

6. RICHMOND SCREW ANCHOR (below) consisting of screw bolt and spiral shell, is held by carpenter foreman C. W. McDANIEL Bolt and shell are cast into pile at lift points, bolt is removed and replaced with eyebolt screw for lifting

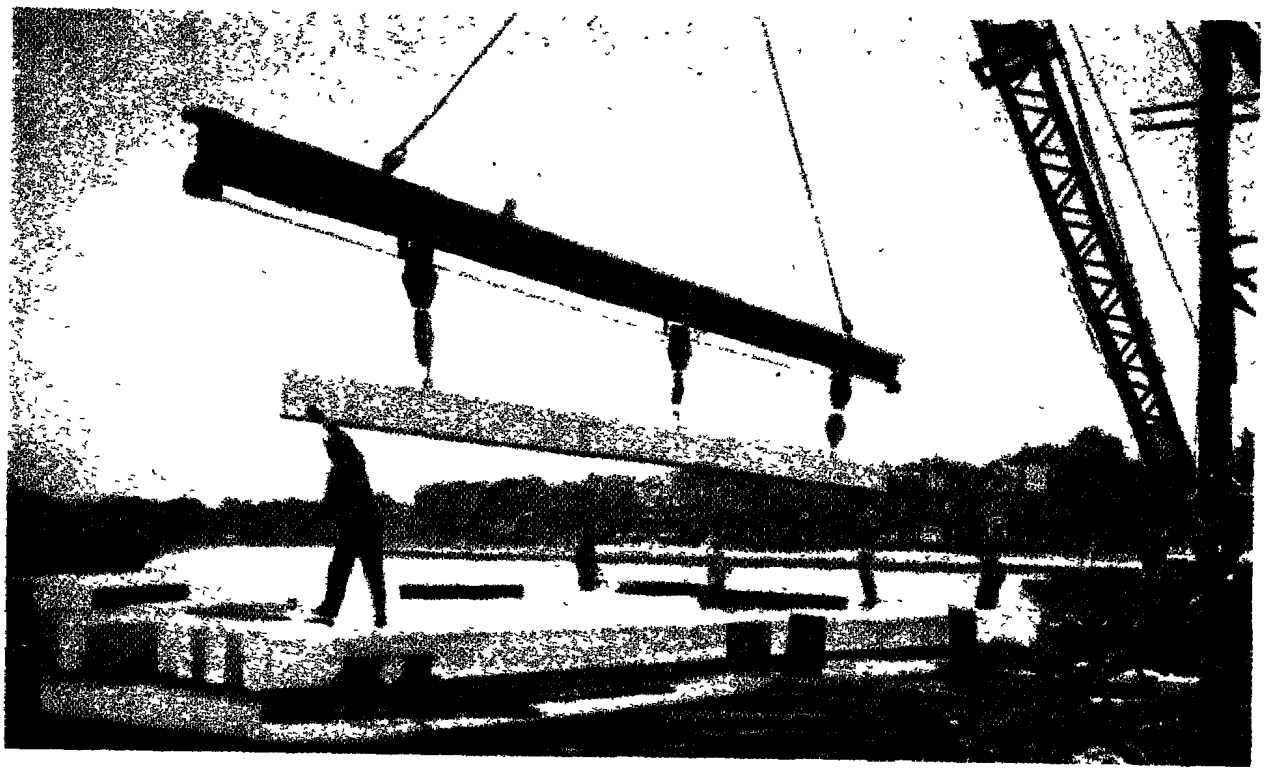


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6





7. STRONGBACK handled by revolving steam traveller lifts piles from platform and places them on barge for transportation to job. Equalization of three point suspension is by dead-ended cable reeved through six multiple sheave blocks.

Intermediate forms for the second group of piles are simple plywood panels with the chamfer moulding nailed in place. They are supported entirely by bearing against the sides of the previously-poured first group For the second stage concrete placing, the runways and working platforms are blocked up on the first group of piles. All contact form faces are carefully cleaned and oiled between uses

All concrete is transit-mixed, delivered to the casting platform in transit trucks by Southern Materials Corp Trucks discharge concrete direct into buggies on the runways for final distribution to the forms The concrete is vibrated with internal-type vibrators and is float finished A liquid curing compound is sprayed on to the top surface immediately after the concrete is finished.

A full-revolving steam crane, mounted on wide-gauge tracks, lifts the completed piles from the platform and moves them either to storage or direct to barges in a slip alongside the casting yard for transportation to the pier job The crane also places the reinforcing cages. It is propelled by a double-drum gas hoist at one end of the crane runway through direct and back-haul hoist lines.

The traveller is equipped with a heavy I-beam strongback from which hang three hooks on multiple-line sheaves reeved up to equalize the lift at all three points This is accomplished by three multiple sheaves fastened to shackles on the strong back, each of which supports another multiple sheave carrying a hook A single wire rope, dead-ended at each end of the beam, runs through all six sheaves; thus the load on each pair of sheaves is equalized.

Each pile is fitted with three equally-spaced lifting points The spiral casing part of a heavy 2-in Richmond screw anchor, held by a screw bolt, is cast into the top face of the pile at each point during concrete pouring When the pile is to be lifted, an eyebolt lag screw is screwed into the incased shell

E C Cunningham is superintendent of the pile casting yard for Tidewater Construction Corp., Norfolk, contractor for the pier job.—(*With acknowledgments to "Construction Methods."*)



8. REVOLVING TRAVELLER also places prefabricated reinforcing cages in forms.

EXPERIMENTAL HOUSES

Nine experimental houses for which the Ministry of Works granted facilities for building are discussed in the Second Report of the Committee appointed to advise the Government on materials and methods of construction All of the types are *fully described and illustrated in the Report* and the Committee's conclusion on each one is given. Three of the types, not previously illustrated in "Building," are briefly described and illustrated here.

Boot "Beaucrete."

THE Boot "Beaucrete" concrete house is erected on foundations of normal strip construction in cast *in situ* concrete A central groove receives the concrete piers and pre-cast footing blocks Framework is formed by twin reinforced concrete piers set at about 3 ft centres. These piers are made up of inner and outer sections bolted together at 4 ft 6 in sections, with a space between to ensure a continuous cavity in the external walls Outer sections are continuous from foundation to wall plate, but inner sections are interrupted at first floor level by a continuous reinforced concrete beam supporting the floor beams This also serves as a lintel for door and window openings

External walls are of cavity construction with an external leaf of 1½ in thick dense pre-cast concrete units, stiffened horizontally by flanges 3½ in wide, which form beds for the horizontal joints and receive the wallties The internal leaf is of foamed slag pre-cast concrete units, 3 in thick, which are rebated at their ends to cover the internal sections of the piers The chimney and major part of the party wall are of cavity brickwork Partitions are of foamed slag concrete slabs

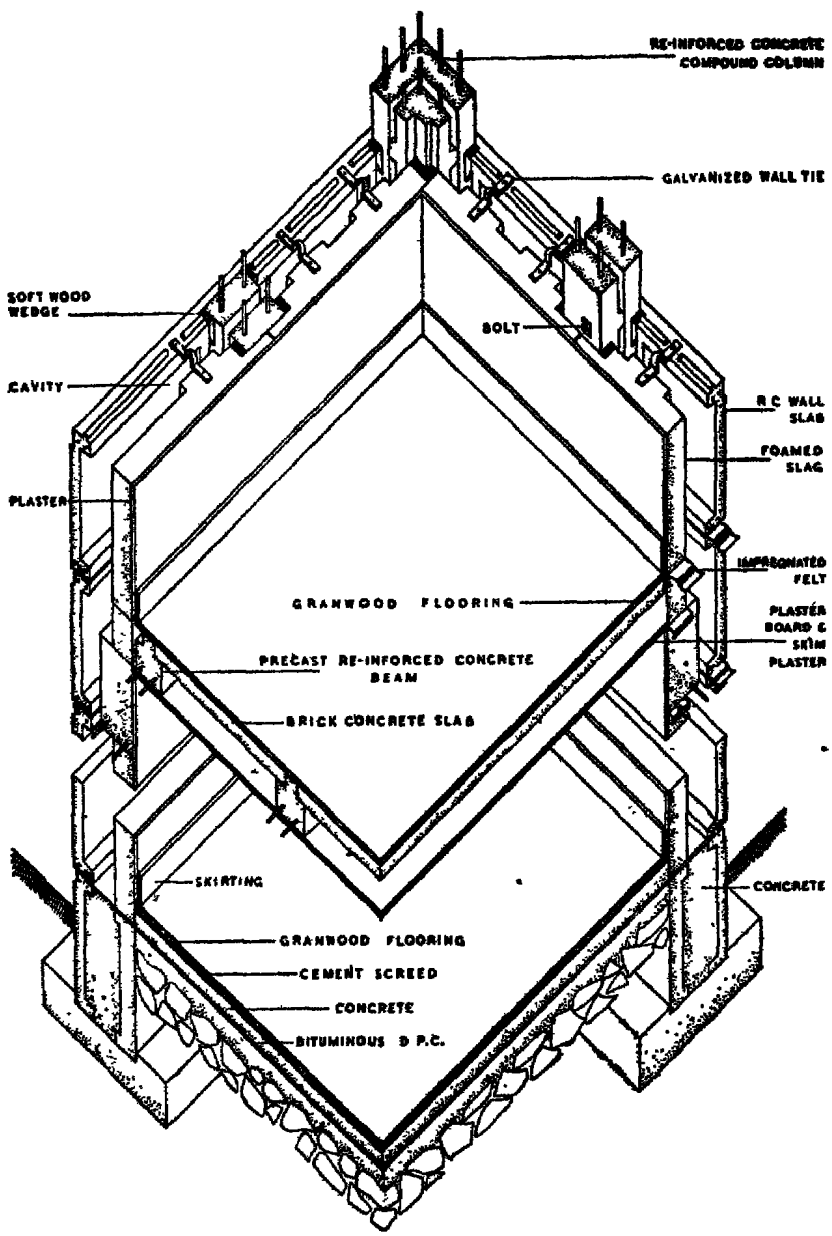
The roof is of heavy framed timber construction covered with plain tiles and lined with hessian-reinforced bitumen felt

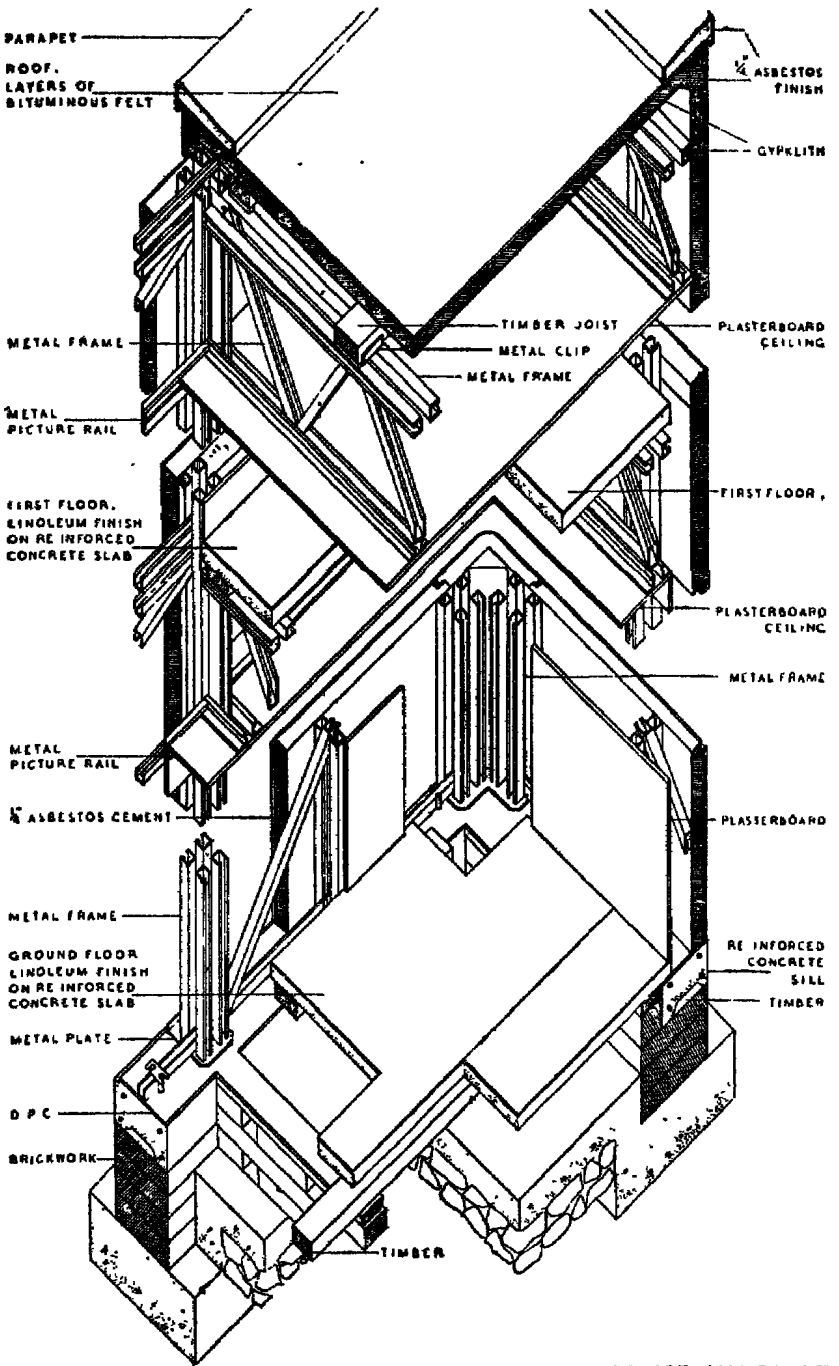
The ground floor has a base of hardcore, a sub-floor of concrete, a waterproof membrane and another concrete slab with Granwood finish The upper floor is of pre-cast reinforced concrete beams supporting reinforced concrete slabs, finished with Granwood All ceilings are of ⅜ in plasterboard, finished with calcium sulphate plaster.

Birmingham Corporation House

The pair of semi-detached houses built by the Corporation of Birmingham are not exactly alike in plan or construction although they have many features in common The main differences are in the external cladding, roof covering and first floor construction House No 1 has two, and House No 2 three bedrooms

House No 1 has foundations of normal mass concrete and a steel frame of proprietary design. This frame consists of stanchions, floor and ceiling joists and roof trusses, formed by welding steel rods, bent to form a lattice pattern web, to flat steel flanges Stanchions and roof trusses are spaced generally at 3 ft centres, but stanchions occur immediately at each side of the party wall and support a roof truss in the cavity.





KEYHOUSE UNIBUILT

The framework is of storey-height braced wall chassis with all trusses spanning the width of the house The sections are 10 ft by 4 ft weighing 65 lbs and are easily handled

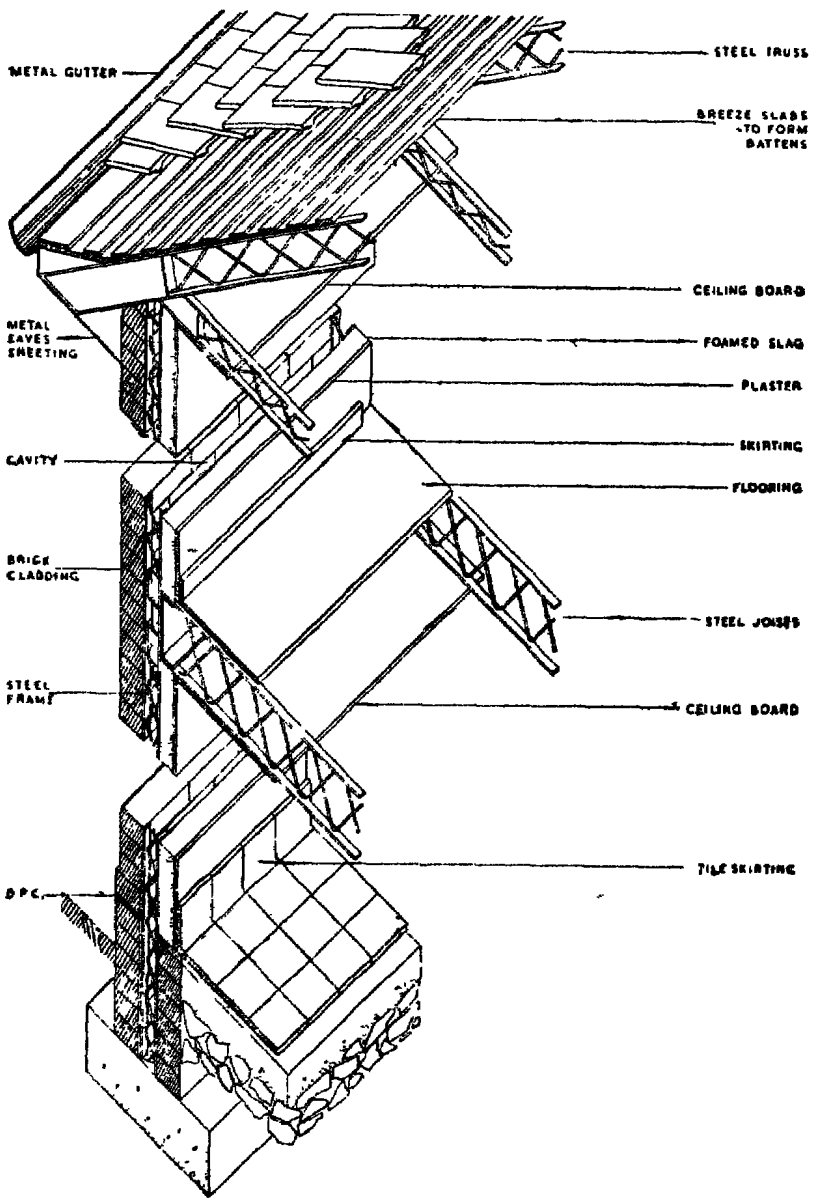
External walls are of 4½ in brickwork, with an internal wall of foamed-slag concrete slab, 3 in. thick, separated by a 3 in wide cavity Both leaves are connected to the stanchions with light metal clips, but felt pads prevent direct connection between the frame and the walls.

In the living room, the floor consists of 6-in hardcore, 4-in concrete, building paper, 2-in. coke-breeze concrete and 1-in. tongued-and-grooved boarding, nailed to the coke-breeze concrete upon a continuous layer of bitumen mastic The first floor is constructed of factory-made panels composed of ¾-in. tongued-and-grooved flooring The panels are rebated at the ends to fit over the flanges of the steel floor joists The panels are generally 7 ft long by 3 ft. wide, but some are made in shorter lengths to conform to planning requirements

The roof is covered with plain tiles laid on pre-cast reinforced lightweight concrete units, which bear upon the steel roof trusses Ground floor ceilings are of ½ in plasterboard, supported by light timber joists, which are covered on their upper surfaces with insulating felt. First floor ceilings are of similar construction, but no cabot quilt is provided.

House No. 2 differs mainly in the following details —On the ground floor the living room and kitchen floors are covered with wood block flooring 1-in in thickness, laid on bitumen mastic Proprietary lapped asbestos sheeting, bolted to the steel frame, is used for the external walls, instead of the brickwork in No 1. The first floor consists of pre-cast reinforced lightweight concrete units of tray shape, rebated to fit on felt pads over the flanges of the steel joists The first floors are finished with coloured asphalt flooring, laid on hard felt For the roof, proprietary asbestos roofing units are used, instead of plain tiles

Stanchions, floor and ceiling joists, and roof trusses are formed by welding steel rods lattice pattern to flat steel flanges The outer wall of brickwork and the inner wall of foamed slag concrete slabs are both connected to the stanchions with light metal clips



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In the prototype houses erected in Coventry and described here, the materials used were those which were available at the time

The framework of braced wall chassis, one storey in height, with all trusses spanning the house width, is made of 18 and 20 s w g. cold rolled strip steel ridge, welded in shop and clipped together at site The sections are 10 ft by 4 ft weighing 65 lb. and are easily handled

External walls are of ¼-in asbestos cement trays, 4 ft by 2 ft by 2½ in with Gypklith wood-wool infilling clipped to the frame with metal keys Gyproc plasterboard was used for the internal lining, ½-in thick and jointed with steel strip For partitions pressed steel studs with ½-in Gyproc plasterboard cladding, 22 s w g steel skirtings, ceiling runners and panel strips were used Party wall was formed by two layers of Gypklith slabs between 5-ply laminated plasterboard.

For the ground floor, pre-cast 4 ft by 2 ft. aerated concrete slabs on timber bearers on sleeper walls were used The first floor was similarly treated but carried by a 20 in. deep lattice girder spanning the house. The roof was treated in the same way as the ground and first floors but finished with ruberoid in two layers, with coping in materials similar to those used for the cladding, laid in bitumen —(*With acknowledgments to "Building"*).

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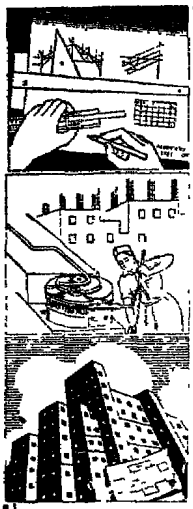
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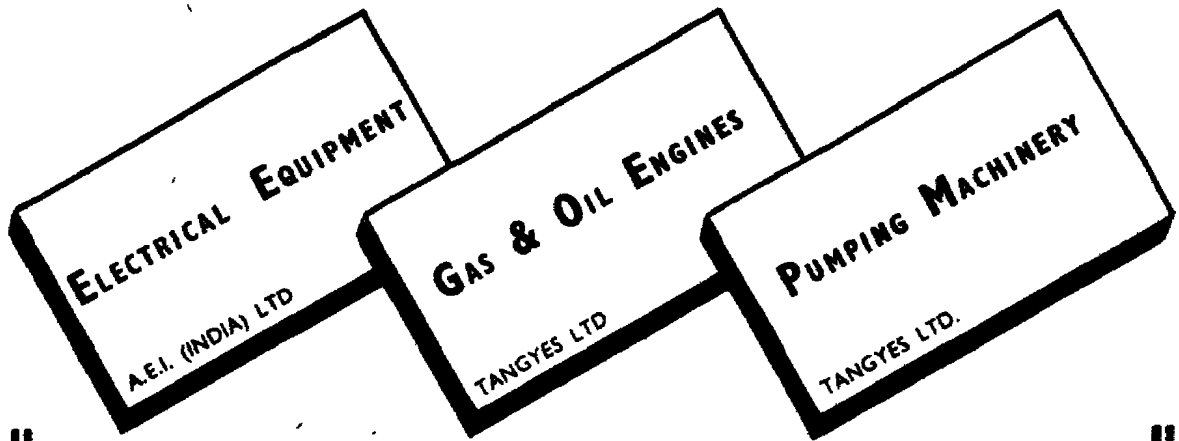
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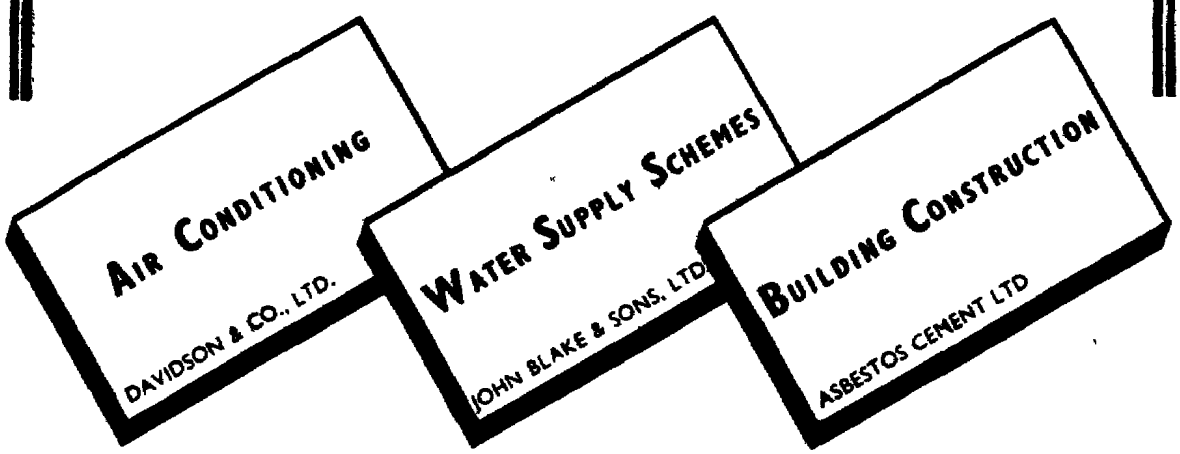
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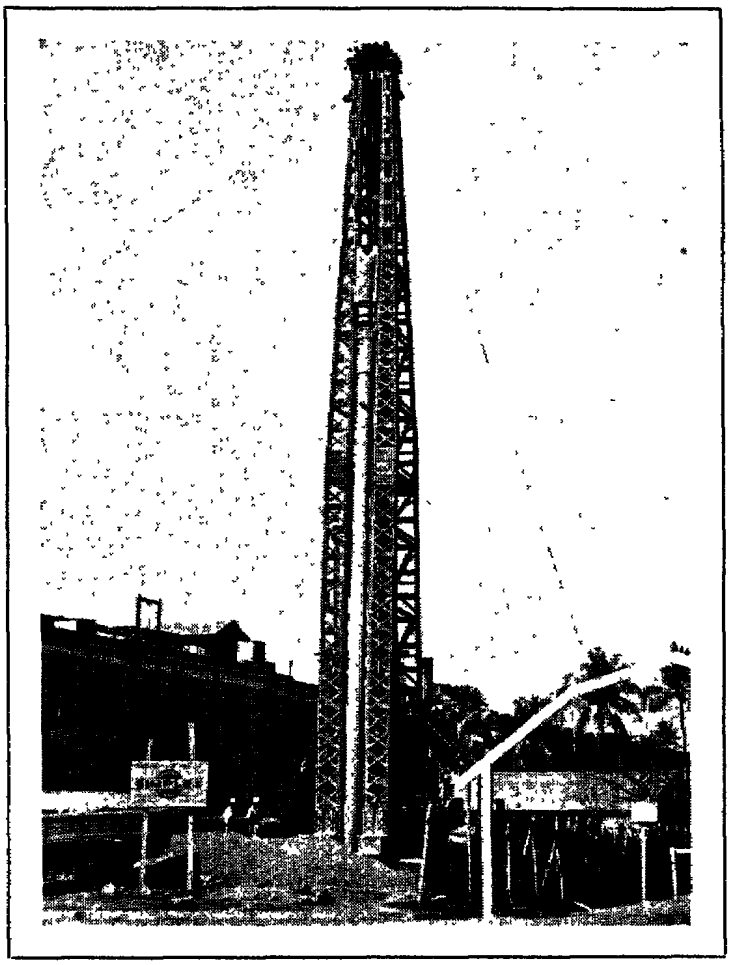
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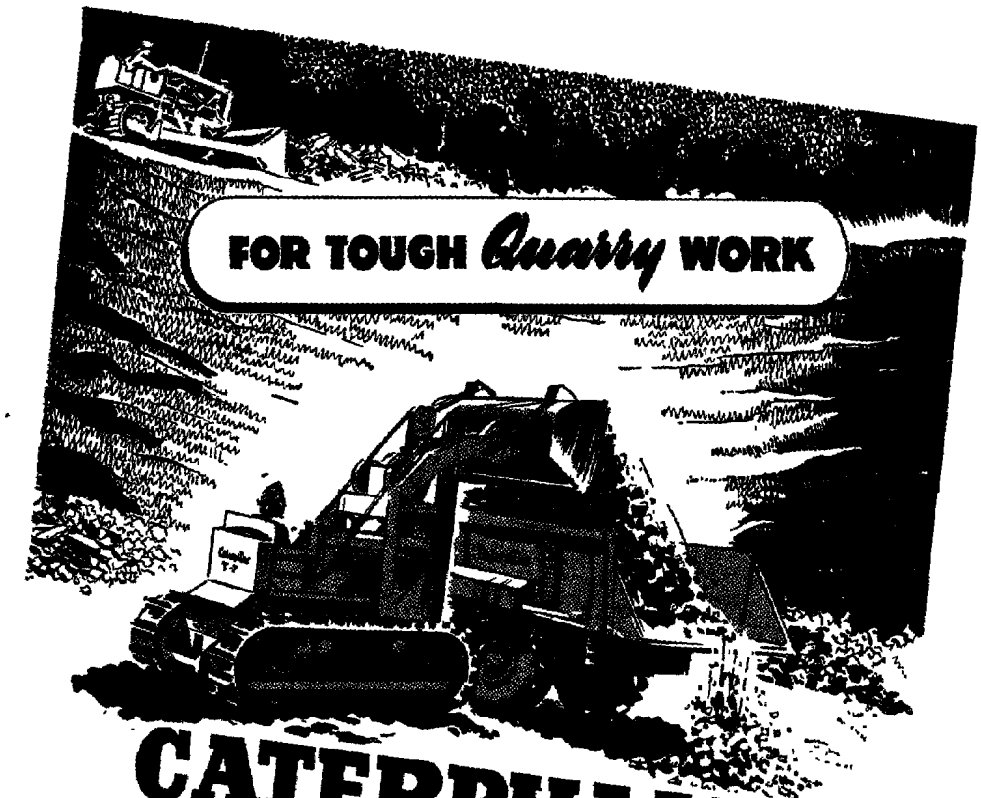
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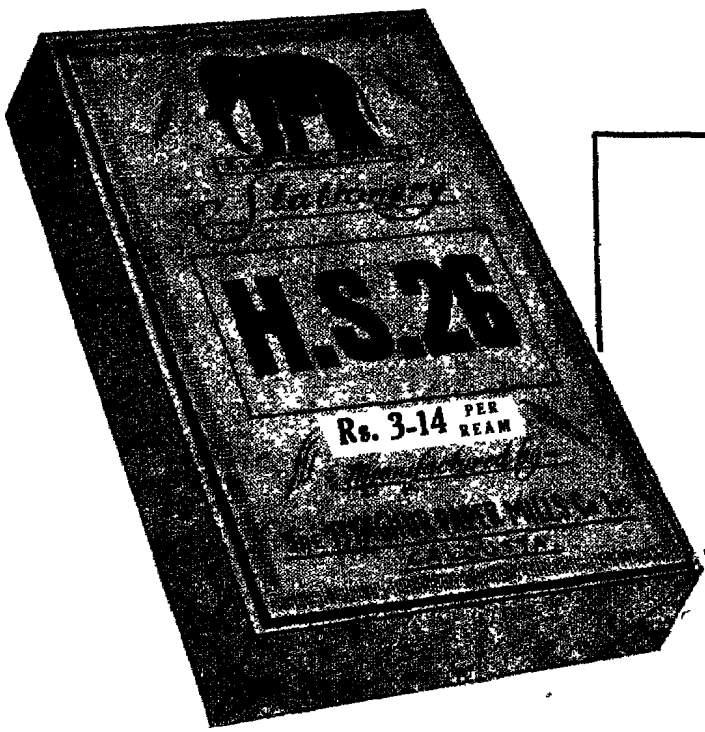
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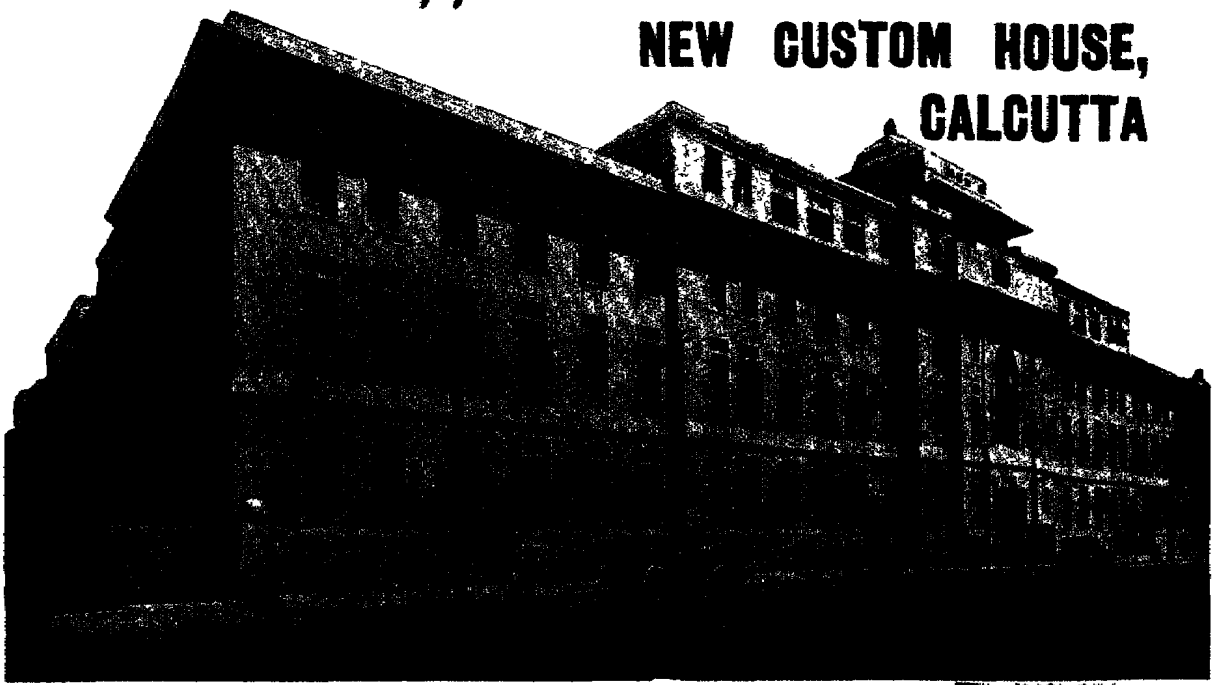
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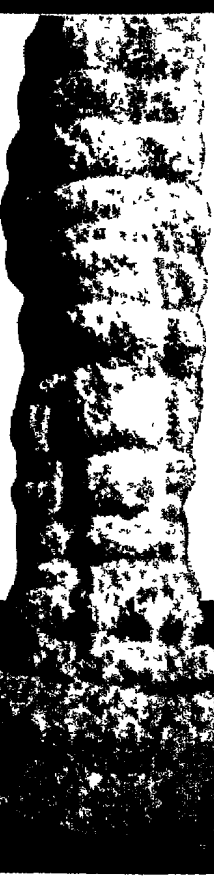
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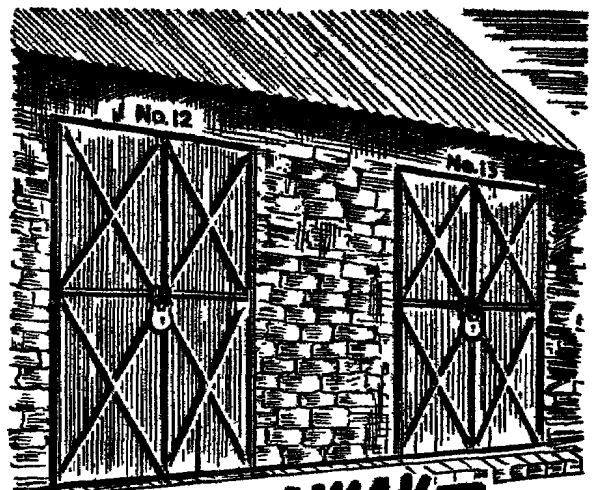
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R S Pathankot
DAULATALA
DASKA
DASUYA
DERA BABA NANAK
DERA ISMAIL KHAN
R S Duryakhan
DERA GHAZI KHAN
DERA NAWAB SAHIB
DARBARSAHIB
DHARAMPUR
DHARIWAL
DHILWAN
DHUDIAL
DHURI
DHABAN SINGH
DINA
DURGAI
DINANAGAR
DINGA
DORAHA
DUNGA BUNGA
DUNYAPUR
EMINABAD
FARIDKOT
FATEHGARH CHURIAN
FAZILKA
FATEJANG
FEROZEPUR CITY & CANTT
GARHSHANKAR
GAKHAR
GAZIGHA
GHAGGAR
GIDDARBAHA
GONIANA
GORAYA
GUJARKHAN
GOBINDGARH
GOJRA
GUJRANWALA
GUJRAT
GUL IMAM
GURDASPUR
GURU HARSHAHAI
GUNJAYAB
HAFIZABAD
HARUNABAD
HARIPUR HAZARA
HASILPUR
HINDUMALKOT
HIRA SING
HOSHIARPUR
HABAPPA ROAD
HASAN ABDAL
ISAKHEL
INJRA
JAGRAON
JAHANGIR ROAD
JAHANIA
JAIJON DOAIA
JAMMU
JALALABAD
JALLO
JAITU
JANDIALA
JANIWALA
JARANWALA
JHANG MAGHIANA
JHELUM
JOGINDERNAGAR
JULLUNDER CITY & CANTT
JUTOGH
JWALAMUKHI ROAD
KHEWRA
KABUL
R. S Chaman & Peshawar
KABUL RIVER
KAMALIA
KALAKE
KALKA
KALABAGH
KALLUR KOT
KAMOKE
KANGANPUR
KANGRA
KAPURTHALA
KARTARPUR
KARUR
KASAULI
R S Kalka
KHAIRABAD
KHUSHAB
KASUR
KHANEWAL
KHAROR PAKKA
KOT RADHAKISHEN
KANDAGHAT
KUNDIAN
KHANNA
KHANPUR
KHASA
KOT ADU
KWARIAN
KILA RAIPUR
KORAT
KOT KAPURA
KOT LAKHPAT
KURALI
LAHORE & LAHORE CANTT
LAKHEWALI
LAKIMARWAT
LALAMUSA
LALIAN
LODHRAN
LOHIAN KHAS
LEIAH
LUDHIANA
LYALLPUR
MACLEOD GUNJ ROAD
MAGHARWAL
MALIKI
MAJITHA
MALAKWAL
MALERKOTLA
MAKHU
MALOUT
MALSIAN SHAHOT
MANDI BAHAUDDIN
MANDRA
MANSA
MARH BALOCHAN
MANDI BUREWALA
MANDI STATE
MANGWAL
MARDAN
MAMU KANJAN
MAUR
MAINWALI
MIRPUR
R S Sarai Alamgir
MIRAN SAHIB
MOGA
MOGHALPURA
MONTGOMERY
MORINDA
MUKERIAN
MUKTSAR (B B &C.I)
MULLANPUR
MULTAN CITY & CANTT.
MURREE
R. S Rawalpindi.
MUZZAFFARGARH
MURIDKE
MIANCHANNU
MINCHINABAD
MIRAN SAHIB
NAKODAR
NANKANA SAHIB
NARANG
NAROWAL
NAWANSHAHR DOABA
NOWSHERA
NURKOT
NURMAHAL
NURPORE ROAD
NAGROTA
OKARA
PALAMPUR
PARACHINAR (*Via Thal*)
PASRUR
PATHANKOT
PAKPATTAN
PATTI
PINDIGHEB
PINJOUR
PATTOKI
PESHAWAR
PHAGWARA
PHILLAUR
PHULERWAN
PIRMAHAL
PIND DADAN KHA
PIPLAN
QAIDAN
QILLA SHEIKHUPURA
QILLA SOBHA SINGH
RAHON
RANBIRSINGHPURA
RAIKOT (*R S Mullanpur*)
RAHWALI
RAIWIND
RAHIMYARKHAN
RAJA JANG
RANBIR SINGHPURA
RORANWALA
RAWALPINDI
RENALA KHURD
RUPAR
RURALA ROAD
SADIKABAD
SAMBRIAL
SAILA KHURD
SANAHWAL
SANGLA HILL
SILLANWALI
SARA ALAMGIR
SARGODHA
SARNA
SATNAUR BADESRON
SHAH ALAM
SHAHDARA
SHARAKPUR (*R. S Shahdara*)
SHAHPUR SADAR
SHAH CHAURASI
SHORKOT ROAD
SHUJABAD
SIALKOT
SIHALA
SIMLA
SIRHIND
SOLON
SUKHO
SULTANPUR LODHI
SUKHEKE
SRINAGAR
R S Havelian or Rawalpindi.
TALAGANG (*R. S Chakwal*)
TALWANDI
TARN TARAN
TANDA URMAR
TANDIANWALA
TANK
TAXILA
THERI SANSI
TEHSIL SHAKARGARH
THABALKE
THAL
TOBA TEK SINGH
VEHARI
VERKA
WAZIRABAD
WARBURTON
WASAWEWALA

NEW DELHI AREA :—

ACHNERA
AGRA
ALAL
ALIGARH
AMROHA
AMBALA CITY
AMBALA CANTT
ATRAULI ROAD
BABRALA
BARHAN
BARARA
BARNALA
BAGHPAT ROAD
BAH (*R S Balangar.I*)
BAHADURGARH
BAHJOI
BARETA
BASIKIRATPUR
BEGAMABAD
BHIWANI
BRUCHCHU
BALLABGARH
BARAUT
BUDHLADA
BULANDSHAHR
BIJNOR
BRINDABAN
CHANDAUSI
CHANDPUR SIAU
CHHINTANWALA
CHOLA
DADRI
DALMIA-DADRI
DIBAI
DEHRA DUN
DELHI
DELHI SHAHDRA
DEOBAND
DANKAUR
DAURALA
DHAMPUR
DHANARI
DOIWALA
ETMADPUR
FARIDABAD
FEROZABAD
GANAUR
GANGOH
GARHMUKTESAR
GHAZIABAD
GHARAUNDA
GOT
GOHANA
GURGAON
GULAOTHI
HALDAUR
HANSI
HARDWAR
HAPUR
HATHRAS
HISSAR
HASANPUR (*R S Gajraula*)
JAGADHRI
JAKHAL
JALESAR ROAD
JAWALAPUR
JIND CITY
JULANA
KALANWALI
KANTH
KANDHLA
KAURARA
KHATAULI
KHEKRA
KHURJA
KARNAL
KAITHAL
KASHIPUR
KASIMPUR KHERI
KOSI KALAN
KOSLI
KOTDWARA
KURUKSHETRA
LEHRAGAGA
LHAKSAR
LOHARU
MANDI DHANAURA
MADLAUDA
MANGLORE
MANSURPUR
MEERUT
MILAK
MORADABAD
MOWANA *MANDI O/A*
MUNDLANA
MURADNAGAR
MURSAN
MUSTAFABAD
MUTTRA
MUZAFFARNAGAR
NAJIBABAD
NAGARIA SADAT
NAGINA
NABHA
NANAUTA
NARELA
NARWANA
PALWAL
PANIPAT
PATIALA
PILKHUWA
RAIWALA
RAJA-KA-SAHASPUR
RAJGHAT-NARORA
RAJPURA
RAMAN
RAMNAGAR
RAMPUR
RAMPUR MANHYA RAN
RAMPURA PHUL
RAYA
REWARI
ROHANA KALAN
ROHTAK
ROORKEE
RIKHIKESH
SAFIDON
SAHARANPUR
SAMPLA
SANGRUR
SASNI
SUNAM
SAMALKHA
SAMBHAL HATIM SARAI
SARSAWA
SEOHARA
SHIKOHABAD
SHAHABAD MARKANDA
SHAMLI
SIRSA
SONEPAT
SIMBHAOLI
TAPA
TAPRI
TARAWARI
THANA BHAWAN
THANESAR
TUNDLA
TOHANA
UCHANA
UKLANA

CAWNPORE AREA :—

ACHALGANJ
ACHHALDA
AGSAULI
AHIMANPUR
AHRAURA ROAD
AJGAIN
AJODHYA
AKBARPUR
ALIGANJ
ALLAHABAD
AMETHI
AMOUSI
ANJHI
ANUPGANJ
AONLA
ASAFFUR
ATARRA
AURAIYA
AURIHAR
AZAMGARH
BABHNAN
BAGHRAWAN
BADLAPUR
BADSHAHPUR
BAHERI
BAHRAICH
BAHRAMGHAT
BAISWARA
BAITALPUR
BAKSHA
BALAMAU
BALLIA
BALRAMPUR
BAMIANA
BANBASSA
BANDA
BANGAR MAU
BARA BANKI
BAREILLY
BARHAJ BAZAR
BARHNI
BARHALGANJ
BASAI
BASHARATGANJ
BASTI
BELLAREIN
BELTHARA ROAD
BENARES CANTT
BENARES CITY
BEWAR
BHADOHI
BHARTHANA
BHATPARRAN
BHATNI
BHITAURA
BHITI
BHONGAON
BIJAURA
BILGRAM
BILPUR
BAINDKI ROAD
BISALPUR
BISWAN
BODARWAR
BRIDGMANGANJ
BUDAUN
BURHWAL
CAMPIERGANJ
CAPTAINGANJ
CAWNPORE
CHANDAULI MANJHWAR
CHANDWAK
CHAURICHURA
CHHITAUNI
CHHIBRAMAU
CHILBILA
CHILH
CHILWARIA
CHIRGAON
CHIT-BARAGAON
CHOWKGHAT
CHUNAR
COLONELGANJ
DARYABAD
DARYAOGANJ
DATIA
DHEENA
DIBIAPUR
DILDAR NAGAR
DOBHI
DOHRIGHAT
DULAHPUR
ETAH
ETAWAH
FARRUKHABAD
FATEHGARH
FATEHPUR
FYZABAD
GAHMAR
GANJDUNDWARA
GAURIGANJ
GHAZIPUR
GHOSI
GHUGHLI
GOLA GOKARAN NATH
GONDA
GORAKHPUR
GOSHAINGANJ
GURSHAHAIGANJ
HAIDARGARH
HALDWANI
HANDIA KHAS
HARDOI
HARGAON
HARNAMGANJ
HARPALGANJ
HARPALPUR

Cawnpore Area (Contd.)

JAHANAGANJ ROAD
JAIS
JAKHANIA
JALALPUR
JANGHAI JN.
JANGIGANJ
JARWAL ROAD
JASWANTNAGAR
JAUNPUR
JEONATHPUR
JHANSI
JHINJHAK
JHUSI
KABRAI
KACHHLA GHAT
KACHHWA ROAD
KADIPUR
KAIMGANJ
KALPI
KAMALGANJ
KAMALPUR
KANAUJ
KANCHAUSI
KARARI
KARENGI
KARWI
KASGANJ
KASHI
KATHGODAM
KATHKUIYAN
KATKA
KAWAPUR
KEORIPUR
KERAKAT
KHADA
KHAGA
KHALILABAD
KHATEMA
KHETA SARAI
KHORASAN ROAD
KICHHA
KIDIHDAPUR
KOHNDAUR
KONRH ROAD
KOPAGANJ
KUNCH
KUNRAGHAT
KUSMHI
LACHHMIPUR
LAKHIMPUR KHERI
LAKSHMIGANJ
LALGANJ
LALITPUR
LAR ROAD
LUCKNOW
MADHOGANJ
MADHOSINGH
MAHMUDABAD
MAHOBA
MAHOLI
MAHPUR
MAINPURI
MAKRANDPUR
MALIHABAD
MALIPUR
MALLAWAN
MANAURI
MANDAH ROAD
MANKAPUR
MANPUR NAGARIA
MAREHRA
MARIAHU
MASKANWA
MAU AIMA
MAU NATH BHANJAN
MAURANIPUR
MEJA ROAD
MIRANPUR KATRA
MIRZAPUR
MISRIKH TIRATH
MOGHAL SARAI
MOTH
MUHAMMADABAD GOHNA
MUNDERWA
MUSAFIRKHANA
NAGARIA SADAT
NAIPALGUNJ ROAD
NANDGUNJ
NANPARA
NAUTANWA
NAWABGUNJ GONDA
NIBKAROR
NOWGAWN
ORAI
ORCHHA
PACHPERWA
PADRAUNA
PARIAWAN
PARTABGARH
PATIALI-ON-GANGES
PAYAGPUR
PEPPERGANJ
PHAPHUND
PHARENDA
PHARIHA
PHEPHNA
PHULPUR
PIPRAICH
PITAMBERPUR
PILIBHIT
PUKHRAYAN
PUNCH
PURANPUR
RAGAUL
RAGHURAJ SINGH
RAI BARELI
RAJA-KA-RAMPUR
RAJA TALAB
RAJAWARI
RAMKOLA
RANI-KI-SARAI
RASAULI
RASRA
RASULABAD
REOTI
RICHHA ROAD
RISIA
RUDAIN
RUDAULI
RURA
SAADAT
SAFDAR GANJ
SAFIPUR
SAHATWAR
SAHAWAR TOWN
SAHJANWA
SAIDPUR BHITA
SAIYEDRAJA
SAKALDIHA
SALIMPUR
SANDILA
SARAIMIR
SARDARNAGAR
SARJU
SHAHGANJ
SHAHGARH
SHAHJAHANPUR
SHAMSHABAD
SHEIKHUPUR
SHIVPUR
SHOHRATGARH
SHRI KRISHNA NAGAR
SIDHAULI
SIKANDARPUR
SIKANDRA RAO
SISOTARGHAT
SISWA BAZAR
SITAPUR
SORON
SULTANPUR
SURAIMANPUR
SURIAWAN
TAHSIL DEORIA
TAHSIL FATEHPUR
TAKIA
TAMKOHI ROAD
TANAKPUR
TANDA
TARAON
TARIGHAT
TILHAR
TIKRI
TULSIPUR
UJHANI
UNAO
UNCHAULIA
USKA BAZAR
WALTERGANJ
YADVENDRA NAGA
YUSUFFUR
ZAMANIA

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"EVEREST" ASBESTOS CEMENT ROOFING PRODUCTS and Accessories, "BIGSIX" Corrugated Grey Sheets, "TRAFFORD" Gray Sheets, Asbestos Wood, etc.

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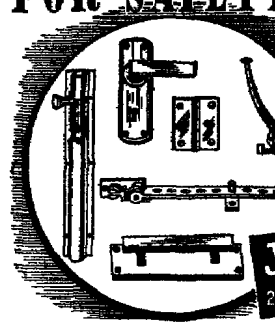
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FOR ALL TYPES OF
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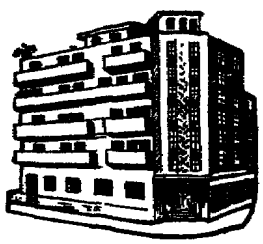
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IN the construction of a bungalow or building, a careful thinking regarding their door and window fittings is essential, though it may seem ordinary. The safety of your building entirely depends upon the quality and durability of these fittings. This is assured if you choose our BRASS FITTINGS.

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THE BOMBAY BUILDING MATERIALS TRADING CO.

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FREELY AVAILABLE AGAIN

Bowranite Anti Corrosive Paint keeps your iron and steel work safe from the ravages of rust and corrosion.

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CONTRIBUTIONS to the Journal in the shape of articles and photographs are always welcome and those that are accepted will be paid for.

AT YOUR SERVICE

NOT ONLY HAVE ALL PRE-WAR LONG DISTANCE TRAINS BEEN RESTORED

BUT

ONE ADDITIONAL MAIL TRAIN HAS BEEN INTRODUCED

B B & C I

MILLARS'



present

ANSON

A NEW

7NT MIXER

★ Re-design Loading Bucket with

★ Smooth, Welded Joints (no sharp angles) giving

★ Rapid, Clean Feed into the Mixing Drum and

★ No Hold-up of Aggregates.

★ New type Discharge Chute providing

★ Almost Instantaneous Discharge

★ Welded Frame greatly reducing Weight.

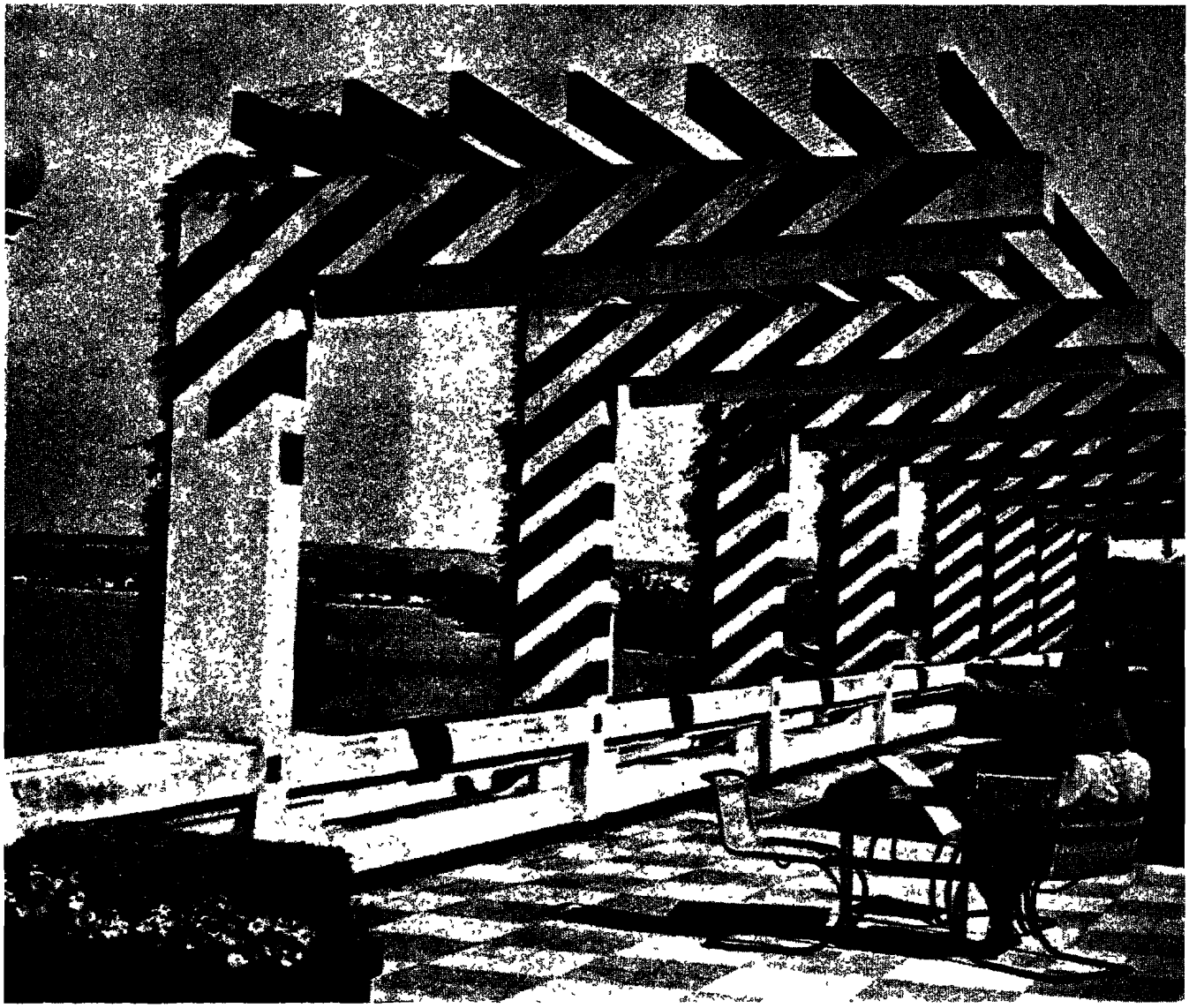
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The beautiful terrace of the Hotel of Puerto Varas, Chile, built for tourists by the Government of Chile.

Make The Home Grounds Beautiful

The grounds around the home can easily be made so attractive that they would be the pride of the owner and the joy of the family. Inviting walks, trim drives, limpid pools and terraces, pergolas and garden accessories such as concrete bird baths, sun dials, benches and vases are possibilities within reach of the average budget.

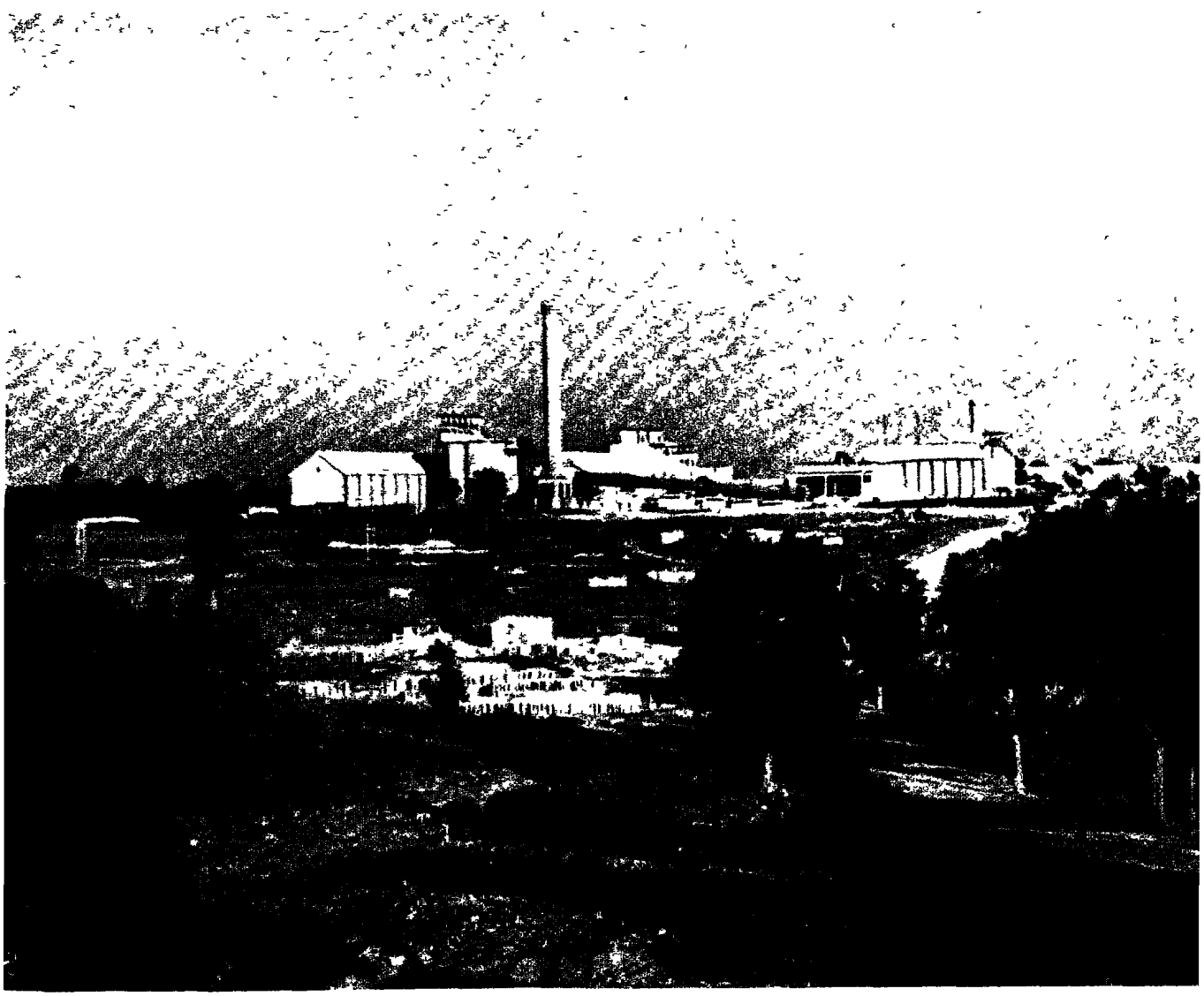
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451-S.1.D.

Regd. No. B. 2422

The INDIAN CONCRETE JOURNAL

VOLUME XX DECEMBER 1946 NUMBER 12



THE CHAIBASA CEMENT WORKS AT JHINKPANI, BIHAR.

(Also see back cover and supplement)

KILLICK, NIXON & CO.

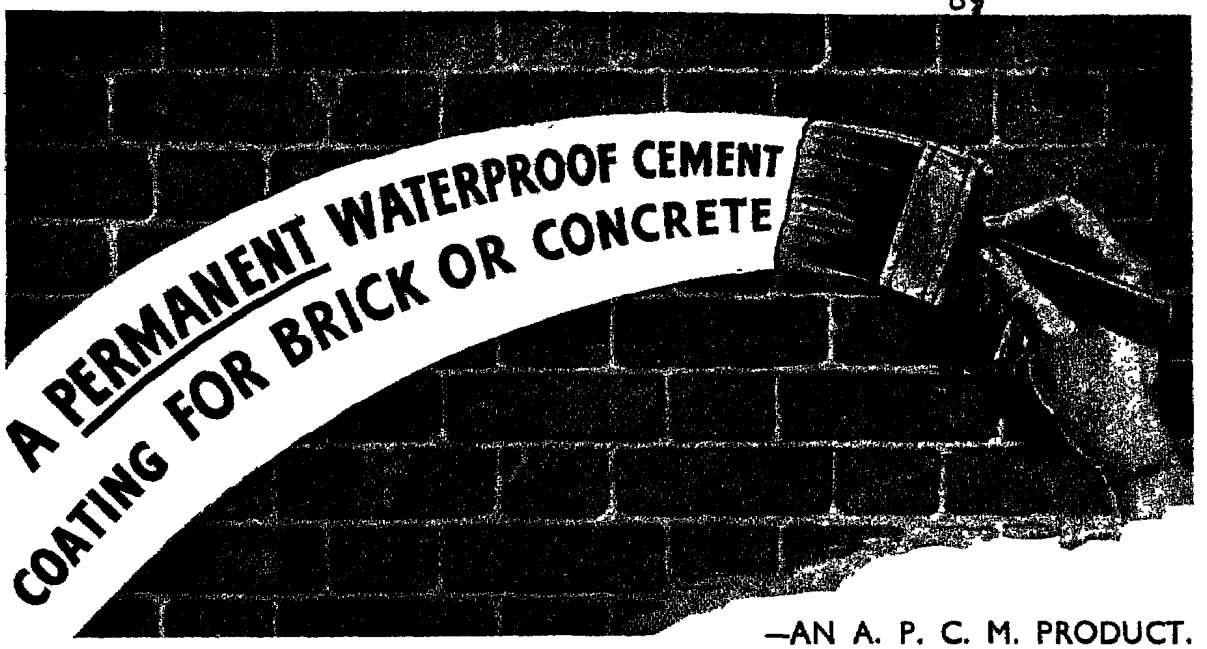
(IMPORT & AGENCY DEPT.)

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CANVAS, TWINE
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(It will be appreciated that some of the above are still in short supply.)

CLEARING & FORWARDING

HOME STREET, BOMBAY.



—AN A. P. C. M. PRODUCT.

HERE is a coating which can be applied to new or old concrete, brickwork, stone, asbestos cement sheets, lime plaster, cement rendering, etc.

It is unaffected by humidity or condensation.

It will not flake, peel or wash off and can be scrubbed repeatedly without harmful effects.

WALLS AND CEILINGS MADE PERMANENTLY CLEAN, BRIGHT AND LIGHT-REFLECTING : 'Snowcem' provides the ideal medium for the interior or exterior decoration of structures such as factories, hostels, stores, canteens, hospitals, schools, garages, etc., where the walls are of exposed brick or concrete.

Its hard, dustless, washable surface promotes hygienic conditions whilst its permanent brightness gives maximum light reflection.

EXTERNAL AND INTERNAL WALLS MADE WATERPROOF AND DECORATIVE : For cottages, houses and similar structures which call for external or internal treatment, 'Snowcem' not only provides an inexpensive decorative medium but also renders the most porous surface waterproof.

ECONOMICAL AND EASY TO APPLY : 'Snowcem' costs but little more than distemper. It is applied with a brush or spray similarly to other paints.

SHADES : 'Snowcem' is supplied in White, Cream and Silver-grey shades and stocks are expected to arrive shortly.

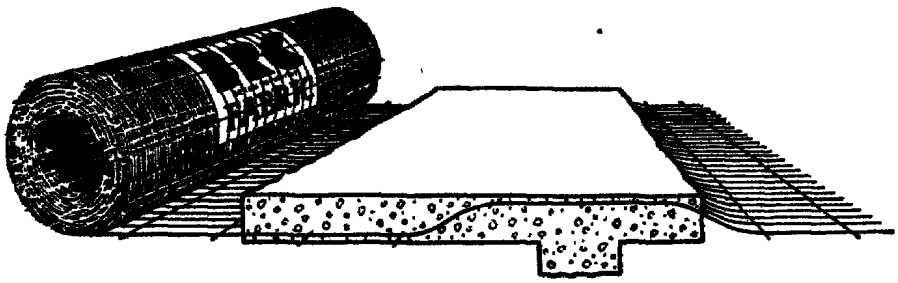
"SNOWCEM"

CEMENT COATING FOR BRICK OR CONCRETE.

KILLICK, NIXON & COMPANY,

HOME STREET, BOMBAY

Uses and advantages of



BRC WELDED FABRIC

SUSPENDED FLOORS AND ROOFS

Reinforced concrete floors carried on walls and beams provide a permanent even surface for the storage of goods, to carry machinery and to form a working area in offices, factories and similar buildings.

The thickness of the concrete and the amount of reinforcement required are determined from a knowledge of the amount and type of loading and the spans between the supporting walls and beams. The simplest form of modern suspended floor construction is provided by concrete slabs reinforced with BRC fabric.

Not only must a floor be able to carry safely the loads which are imposed upon it ; in addition, it should be vermin-proof, waterproof, sound resisting, and it must be able to prevent the rapid spread of fire from floor to floor. The solid concrete floor reinforced with BRC fabric possesses all these qualities and in addition adds a valuable stiffening to the structure as a whole.

Reinforced concrete floors are best used in conjunction with reinforced concrete beams, but they may be conveniently supported on steel beams, which are usually encased or haunched with concrete.

The BRC fabric is arranged so that the main wires run across the slab at right angles to the beams There is usually a sheet placed half an inch above the soffit and shorter sheets near the top of the slab over each interior support. Alternatively, the fabric may be waved so that it is in the bottom of the slab at midspan, and near the top over the supports. In the former case, the reinforcement is supplied in sheets and in the latter it comes to the site in rolls.

The formwork which supports the wet concrete until it has set is made up from timber, steel sheets and angles, or cement-asbestos sheets. When the beams are of reinforced concrete the bearers for the floor forms are usually supported on the beam-side forms. When encased steel beams are used the construction may be similar, but it is customary to suspend the formwork from the beams when these are of steel sections.

It is important that the concrete be mixed without excess water coming up to the screeded surface of the work, so that the full strength and hardness of the concrete may be obtained.

BRC

for

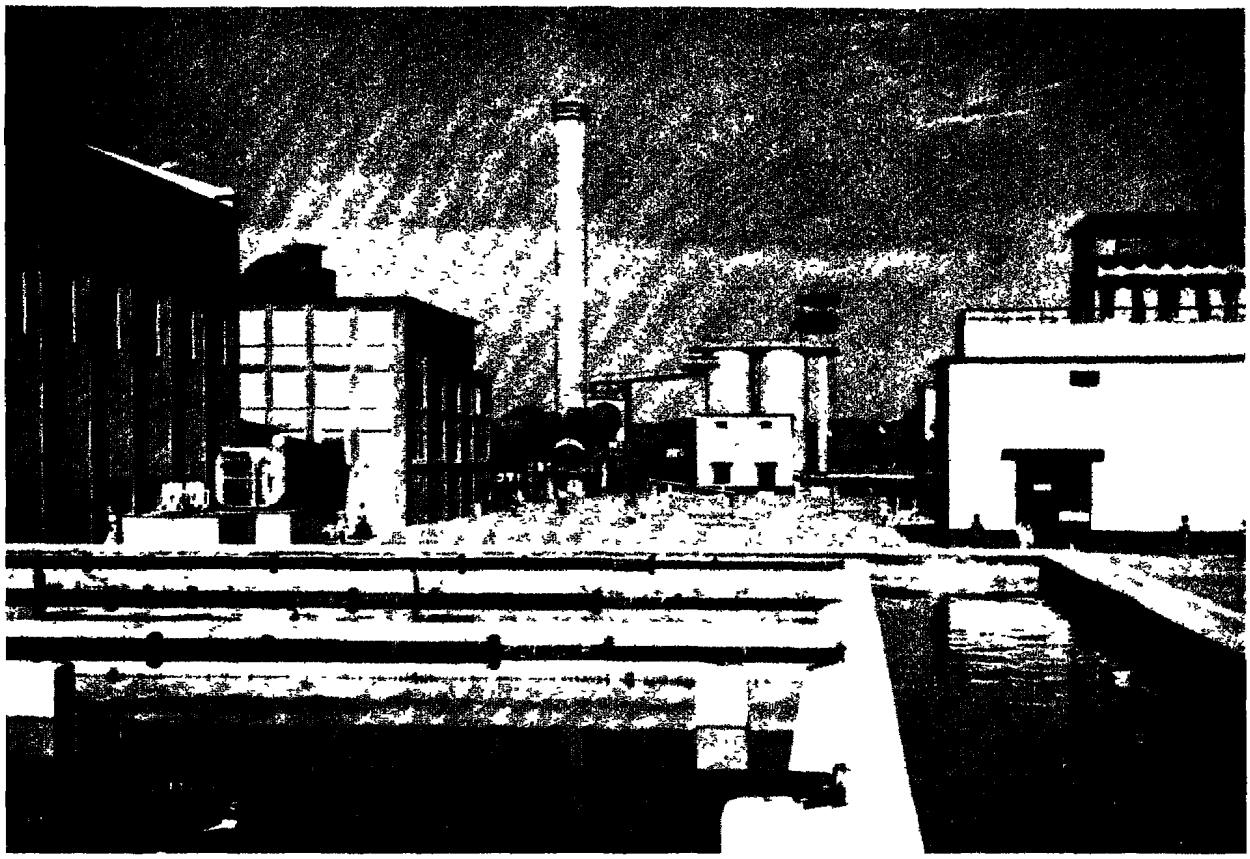
concrete reinforcement

HEATLY & GRESHAM

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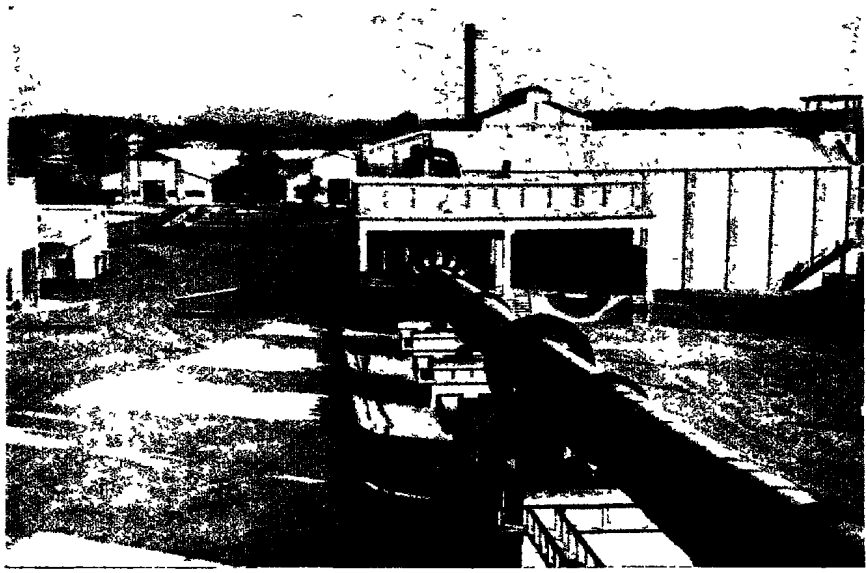
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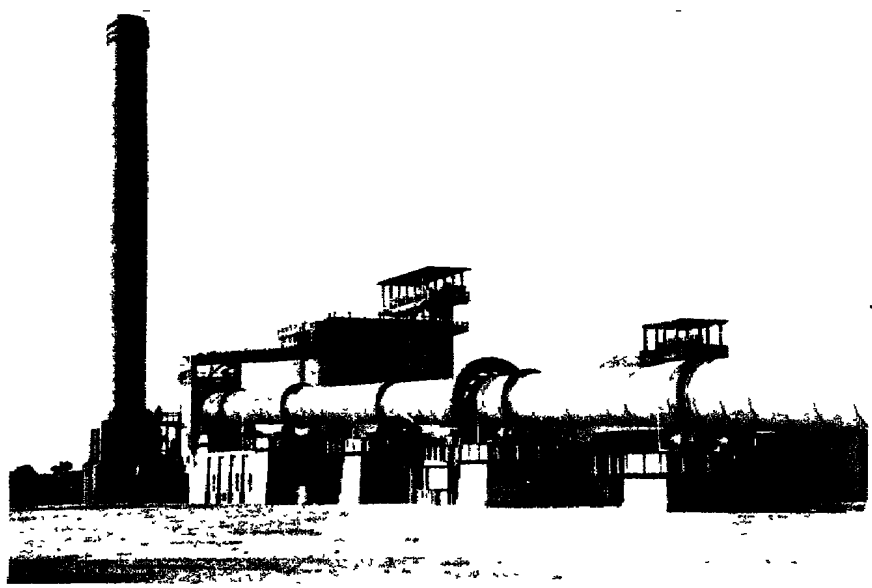
THE CHAIBASA CEMENT WORKS. –A general view

To their vast network of Works operating in the various parts of India, the Associated Cement Companies, Ltd. have just added one at Jhinkpani, near Chaibasa, which was inaugurated by His Excellency Sir Hugh Dow, K.C.S.I., C.I.E., the Governor of Bihar, on Tuesday, the 5th November.

Sir Homi Mody, K.B.E., the Chairman of the Associated Cement Cos., Ltd., was present to welcome His Excellency as well as a prominent gathering of distinguished visitors.



View showing the kiln, with the burners' platform in the background.



Note the 170' high Concrete chimney.

Address Delivered by Sir Homi Mody, K.B.E., the Chairman of the Associated Cement Companies, Ltd., at the Opening Ceremony of the Chaibasa Cement Works on 5th November.

"YOUR EXCELLENCY, LADIES & GENTLEMEN,

My colleagues and I have great pleasure in welcoming Your Excellency and Lady Dow We are grateful to you and your predecessor in office for the encouragement and help given to us The Officers of your Government have also rendered us valuable assistance from time to time, and we take this opportunity of making our acknowledgments to them We record with deep regret the passing away, less than a week ago, of one of them, Mr D P P Singh, Additional Deputy Commissioner, Singhbhum, who had been extremely helpful to us It is a tragedy he should not be here to see the full flowering of this enterprise

Permission for the erection of the Factory was granted by the Central Government in December, 1943, and the acquisition of the site was completed in September, 1944 Owing to the undulating nature of the land, it was not easy to find a suitable location for the Works Nature's planning, so often arbitrary, has been responsible for the fact that the Factory, the Quarries and the Station are all at different levels.

In the course of construction we have had a number of teething troubles On account of various war time controls on materials and transport, progress was impeded at every stage. Skilled labour was scarce, as aerodromes and other military works were going up in the neighbourhood Railway materials, such as rails, were also difficult to obtain, and



The arrival of Their Excellencies Sir Hugh and Lady Dow.



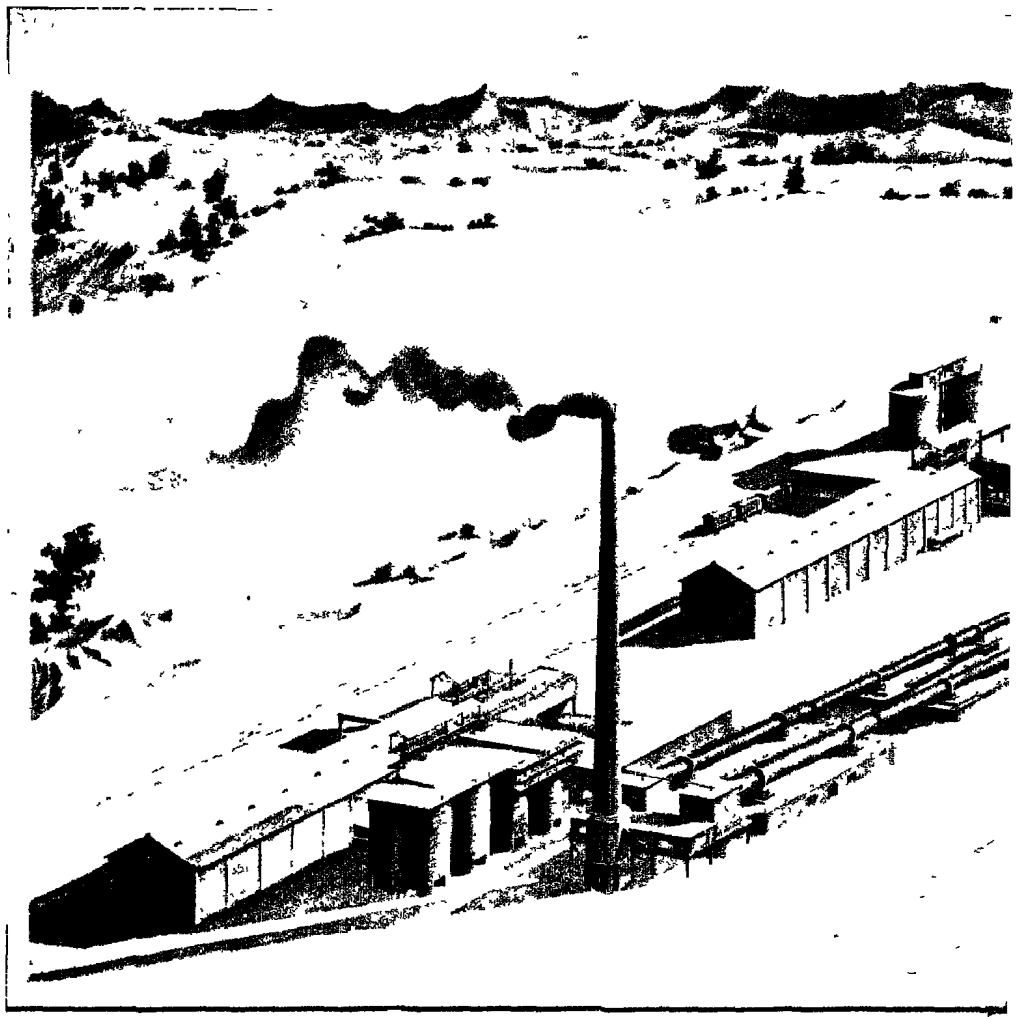
H. E. Sir Hugh Dow addressing the gathering.

this involved carting by road of all the construction materials and a great part of the heavy machinery

Construction commenced in the latter part of 1944 and was completed in 2 years and 2 months Having regard to the difficulties which were experienced, it is an achievement which reflects great credit on our Manager, Mr Turner and his staff They have worked long and hard—not for them the 8-hour day—and their part of the job has been performed according to schedule This is an eventful day for them, and we can appreciate with what satisfaction they must be contemplating their handiwork Where so many have laboured, it is difficult to single out any, but I would specially mention Mr Brandt, Mr Burgess and Mr Tongaonkar, who have also been very helpful

I would like to refer in this place to our Technical Adviser, Mr Moolgaokar This Factory is, in a sense, a monument to his initiative, drive and capacity for organization His work has been of an outstanding character, and it is difficult for me to speak too highly of it All that I would say is that in Mr Moolgaokar we have a top ranking administrator who can take his place by the side of any expert in the line, whether in this country or abroad

I would like to mention an interesting feature in connection with the building of the Factory Most of the unskilled labour has been provided by women, and we could not have had more cheerful and energetic workers Compared with their westernised brothers, I can only envy the lot of the men in this part of the country If all the militant fighters for women's rights, taking a leaf out of the book of the female population of this district, were to stop their misguided activities and concentrate on making woman the bread-winner



An artist's impression of the g

of the family, how much nearer our heart's desire could society be remodelled'

It would lead to a lot of complications if I did not mention here the part played by Mrs Turner and Mrs Moolgaokar in the preparations for this function They have scrubbed the floor, polished the handles, sewn the curtains and done a hundred other odd jobs for the reception of our visitors, and I would like to express our grateful thanks for all they have done It is obvious there is something in the atmosphere of this place which makes women work

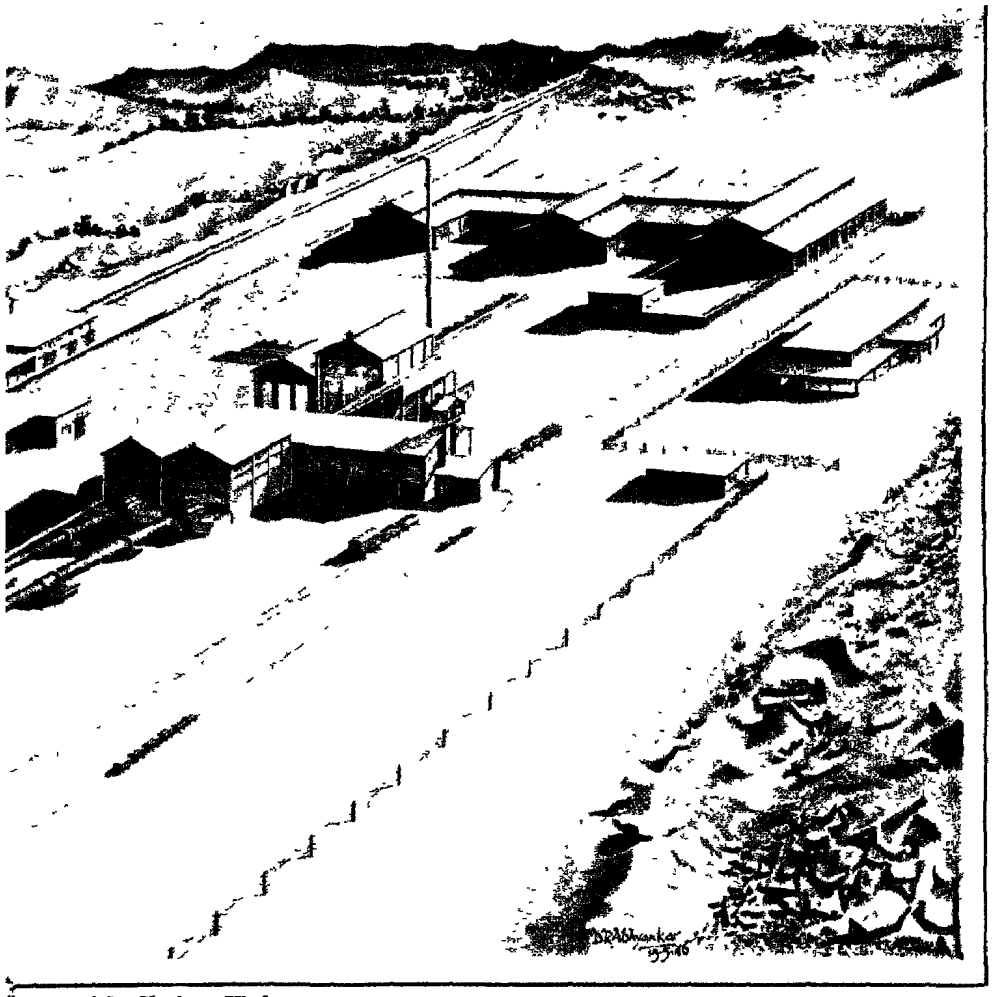
A great part of the machinery and equipment for this Factory was manufactured at some of our other Works, and but for that and the set-up of our organization, it would have been impossible for the Factory to be in production today.

When the ACC came into being in 1936 and for sometime thereafter, everything in the way of plant and equipment had to be imported from abroad In the course of the last few years, however, we have been able to establish at our various Workshops up-to-date facilities for the design and manufacture of the type of cement machinery which our experience has shown to be most suitable and efficient for the conditions existing in this country, and we have raised a body of skilled workers for the purpose Among other advantages, this has enabled us to standardize the various units, thus simplifying problems of construction and maintenance In the layout of this plant, no trouble and expense have been spared to obtain a factory, which will enable us to manufacture a consistently high grade product at the lowest operating cost, while, at the same time, providing the best possible conditions of work for our operatives

As you go round the place, the special features incorporated in the plant will be pointed out to you In course of time, we have learnt to manufacture almost anything from a pin to an elephant, and I hope you will be interested to see the range of our products, from the tiny sewing machine, which has been specially designed for stitching jute bags, to the huge rotary Kiln, which is one of the largest single pieces of machinery in the world

The plant is now capable of turning out one lac tons of cement a year, and it is hoped to duplicate it before long Most of the structures for the duplication have already been completed, and only the main machinery needs to be erected

We hope we can claim that we have made the desert blossom. For our staff and workers it is our aim to provide amenities of a high standard, and particular attention will be paid to housing the unskilled operatives. Our colony is by no means completed, and a great deal



Layout of the Chaibasa Works.

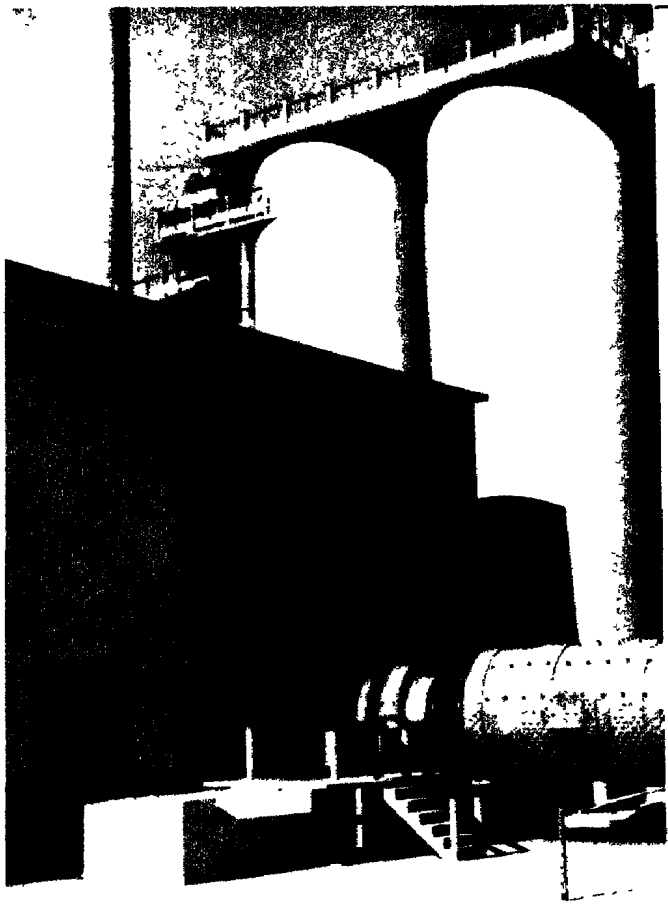
has yet to be done in the way of quarters, Canteens, Rest sheds, Hospital, School, Club and the like. Apart from those who find employment at the Works, we have every hope that the country side generally will benefit from our operations.

We are not unmindful of what we owe to our neighbours, the inhabitants of the surrounding village, and you will be glad to know that the Board of the ACC have sanctioned a capital expenditure of Rs. 30,000 for the provision of a piped supply of fresh drinking water to an overhead concrete tank to be erected in the village, and of electricity to brighten their homes and to help them in their cottage industries. We are also distributing this afternoon cloth to each and every one of our workers in token of our appreciation of what they have done

A great deal has lately been heard about the place of Industry in this country and the measures by which it needs to be regulated A lot of loose thinking and talking seems to be in evidence on the subject While I have no desire to embark on a controversial issue, I would like to express my conviction that there is no real conflict of interest between Agriculture and Industry, and that in the framing of an all-India policy, it should be possible to harmonize the interests of every section of the people, and to provide a framework within which the initiative and enterprise of the individual may be made to serve the best interests of the State

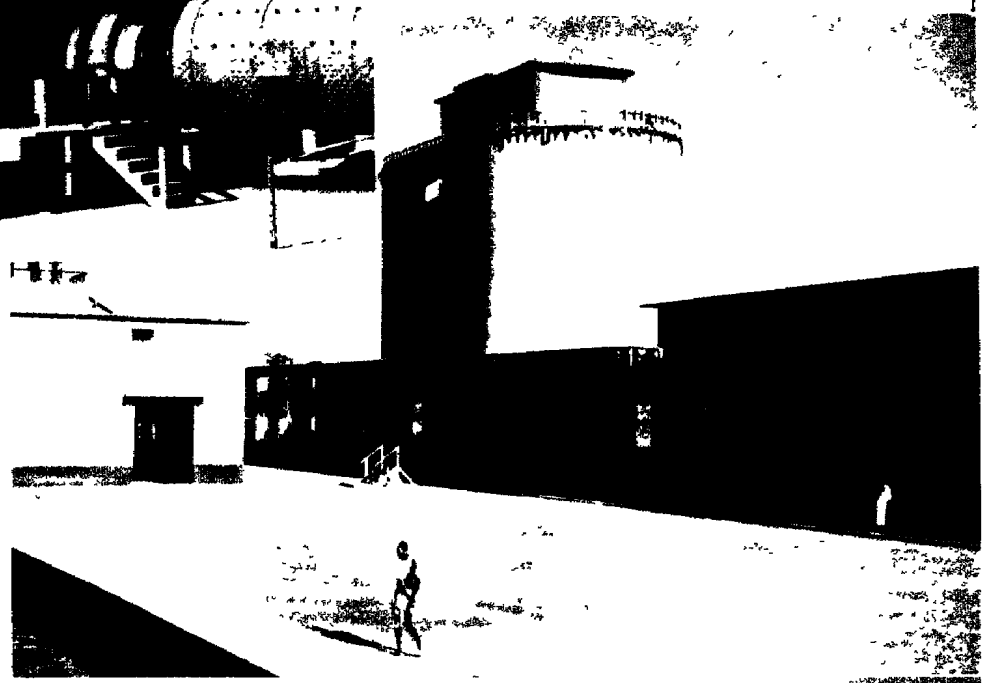
I hope I may without impropriety conclude on a note of what the ACC has been able to achieve in the very difficult years through which the country has passed By the maintenance of our Works at the highest pitch of efficiency and the enlargement of their capacity in the face of various adverse factors, without much in the way of assistance from Government, we have been able to meet very largely the heavy demands made upon us for a product so essential to the prosecution of the war What I would specially like to emphasise in this connection is that there has never been anything like a Black Market in our cement, and we have supplied our products to Government and the public at a price which has actually given us a smaller margin of profit than realized by us in a normal year before the war I do not know how many enterprises can claim such a measure of forbearance, and I hope that in assessing what Industry in this country has done at a critical period, the part played by the ACC may receive some recognition

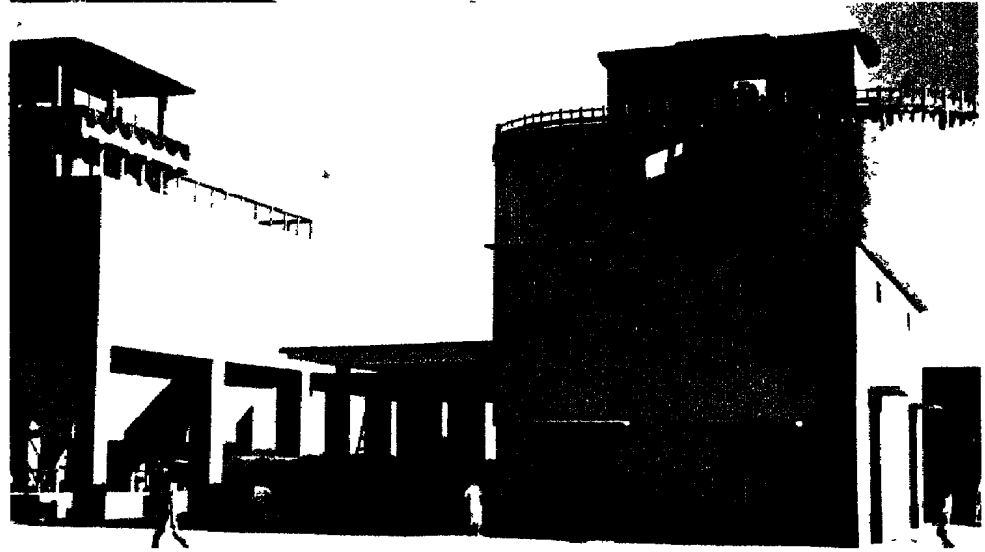
I thank Your Excellency for your presence on this occasion, and have much pleasure in asking you to press the button before you and to set going the wheels of this new enterprise "



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His Excellency Sir Hugh Dow's Reply :

"I appreciate very much the honour and privilege of being invited to perform the opening ceremony of the Chanbasa Cement Works. It is not my first visit to the Works for I came here last August. At that time it seemed to me that there was a tremendous lot still to be done before the Works could be completed, and although Mr Turner assured me that they would be ready to be opened at the beginning of November, I admit that I went away rather sceptical about it. My own experience has so often confirmed Dr Johnson's ponderous pronouncement that 'perhaps an extensive and multifarious performance was never affected within the term originally fixed in the undertaker's mind', and that 'He who runs against Time has an antagonist that is not subject to casualties'. I am, therefore very glad to acknowledge my error, and to endorse the tribute which you have paid to the energy and resource of your manager Mr Turner and his staff.

"The fact that so much of your plant, machinery and equipment has been manufactured in your own workshops is a thing that you may justly be proud of, and I hope that, in spite of the somewhat remote situation of these Works, it will lead to them being something of a show place for those who want to see what India is capable of in this direction. Necessity, they say, is the mother of invention, and during the early war years it was part of my task not only to help divert a good deal of peaceful industry into warlike channels, but to stimulate the indigenous production of a great many things, large and small, which it had hitherto been too readily assumed that India could not make,—or it was not worth while making—for herself. It is a source, therefore, of some satisfaction to me now to see this process in reverse, and to find that the experience and resource, gained under the stress of war necessity, is now being utilised for the purpose of developing Indian industry with greater efficiency, and at a more rapid rate, than might otherwise have been possible.

"I noted with particular pleasure and interest, when I was last here, that in spite of your preoccupation with getting the actual Works going at the earliest possible date, an equal attention was being devoted to the provision of housing accommodation, and social and other amenities, for the staff and workers, and particularly for the unskilled workers. (By the way, aren't we a little too off-hand in what we classify as unskilled and semi-skilled labour? I've seen a great many labourers doing such work with an amount of skill and dexterity which I am sure I could never have acquired even after years of constant practice.) Too often enterprises of this kind start as they did in England at the time of what is now called the Industrial Revolution—with merely getting the Works going with the workers living in hovels, and leaving them to extort, by the slow process of agitation and strikes, such improvements in conditions as they can wring from a reluctant Board of Directors whose main interest is in dividends. We have now reached, and I for one am glad that we have reached, a stage of development in the world's history when such an attitude on the part of any business directorate is crass folly. I believed that in spite of the enormous development of mechanical contrivances during the last war, it has shown us that the *man* is *more than* the machine. Men are terribly handicapped without the machine, but without the man the machine is not only completely useless, but is a source of danger, and if those responsible for our defence have not learned that the welfare and morale of the men must be its prime consideration, then heaven help us in any future war, for we shall be able to do little enough to help ourselves. And this is just as true of peace as it is of war.

"You, Sir Hormusji, have reasonably referred to the loose thinking and talking which goes on about the place of Industry in a predominantly agricultural country. It is less appropriate for me than for you to discuss controversial issues, but I am tempted to ask why it is we are content to refer to India, which is unable to feed itself, as a predominantly agricultural country. The answer is, I suppose, that a relatively large proportion of its population is engaged exclusively in agricultural pursuits.

"Now I have, in the course of a long service in this country, been over a great many industrial concerns, and have spent long years in the detailed examination of agricultural conditions There is one very striking difference between the two pursuits Whereas in industry everyone, from the managing director down to the least skilled operative, puts in a very strenuous day's work all the year round—and I should find it very hard to decide who is the more tired and exhausted at the end of the day—Sir Hormusji Mody or the man who has been shovelling coal into one of his furnaces,—when one considers Agriculture, the most obvious fact is that there are an enormous number of persons, both at the top and at the bottom, who are expecting to get a full day's pay for something which is very much less than a full day's work

"This is not the case in any country which has a balanced economy, whether that country is mainly industrial or agricultural Indeed, we have got to get out of the habit of contrasting Industry and Agriculture, and to realise that agriculture is India's biggest industry, and must therefore be run on business methods No factory could run efficiently with the proportion of half-timers and quarter-timers which are found almost everywhere in agriculture, and India will never be able to feed herself until the causes which lead to this have been removed, whether they are unfair settlements, subinfeudation, the lack of fertilizers or bad marketing arrangements.

"This is not, I think, altogether a digression, because you will depend largely for your labour on the surrounding countryside, and the regularity and steadiness of your workers will to a great extent be a reflection of agricultural conditions in the neighbourhood. I don't know that I would go so far as to say, with your Chairman, that you have made the desert blossom But I will certainly agree that in these Works you have a most promising bud, and will express my confident hopes that in the near future this bud will not only burst into blossom, but will produce rare and refreshing fruit

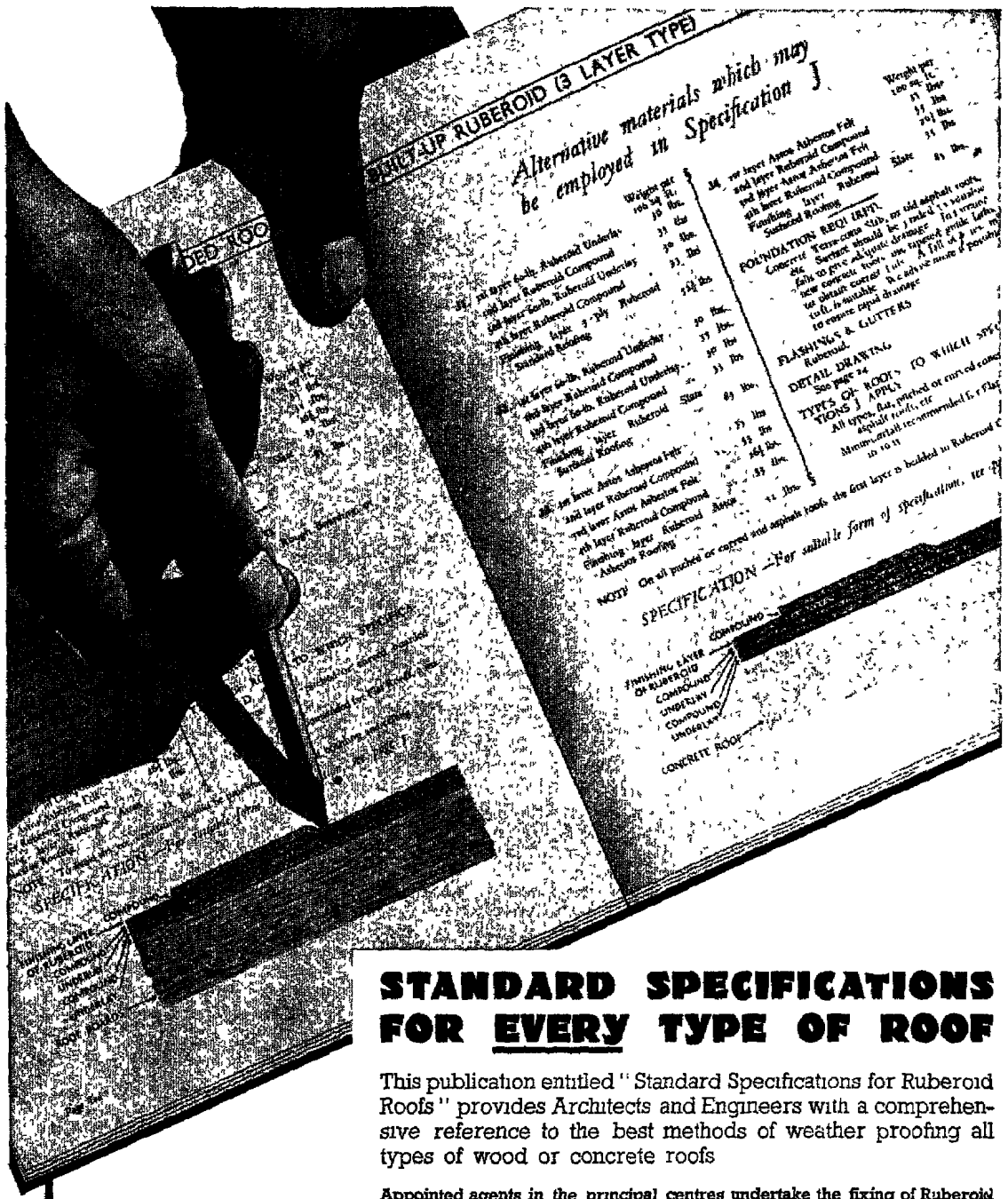
"I now have much pleasure in declaring these Works open, and in wishing success and prosperity to all from the top to the bottom, who have been associated in this splendid enterprise"



Tea party to the staff and their families.



His Excellency saying good-bye to Lady Mody.



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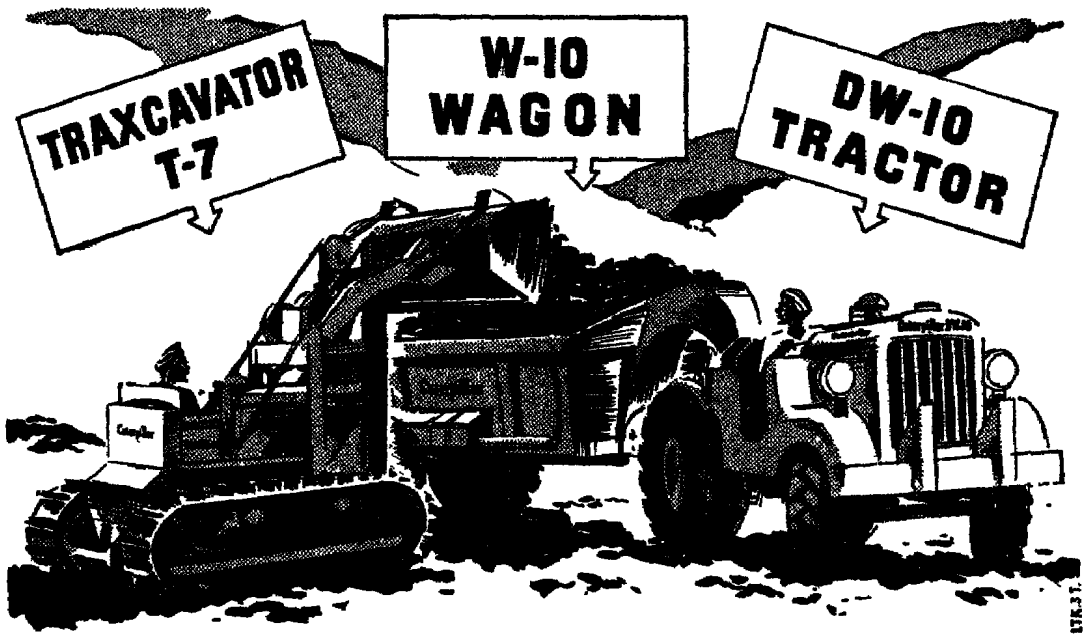
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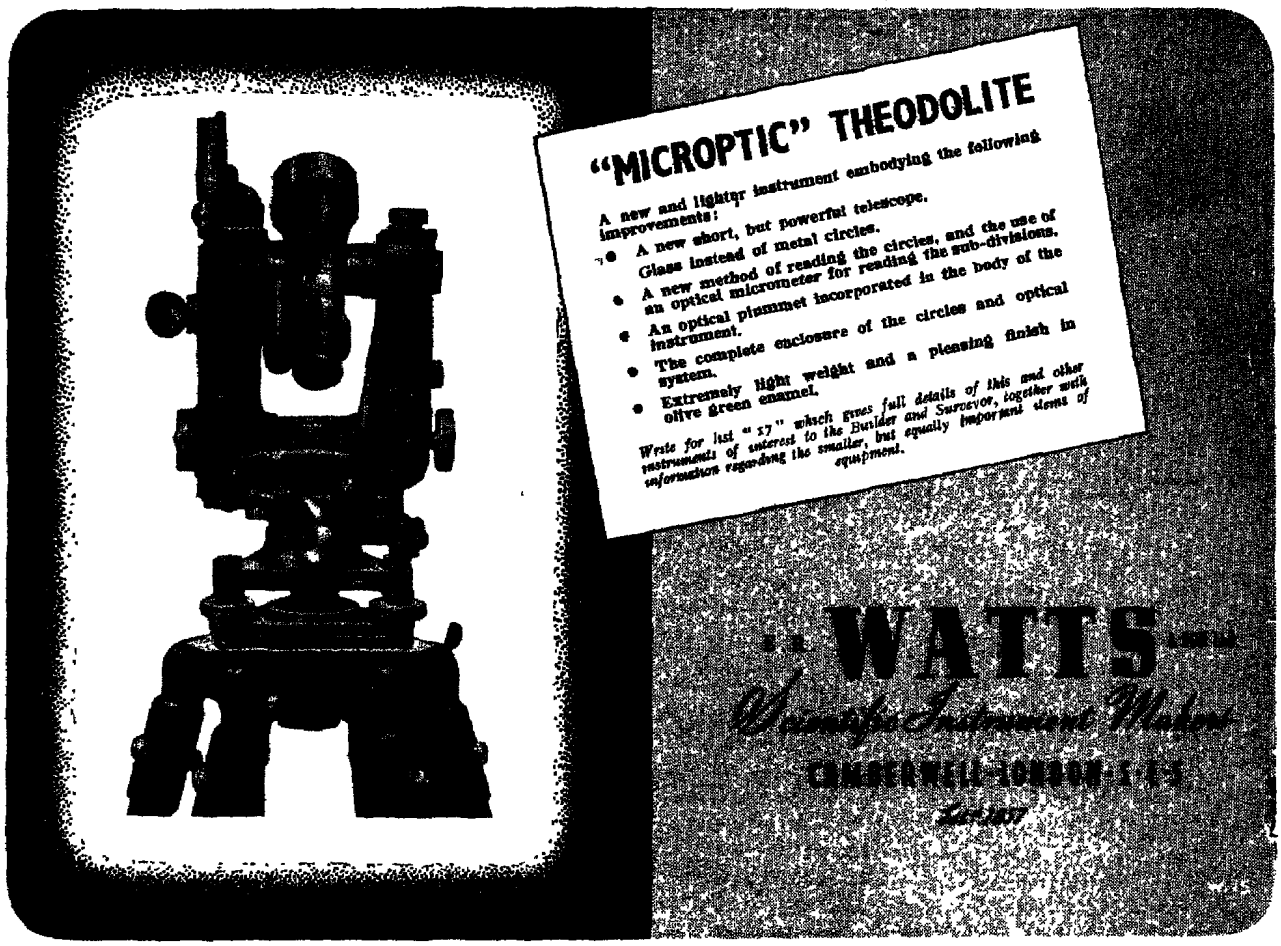
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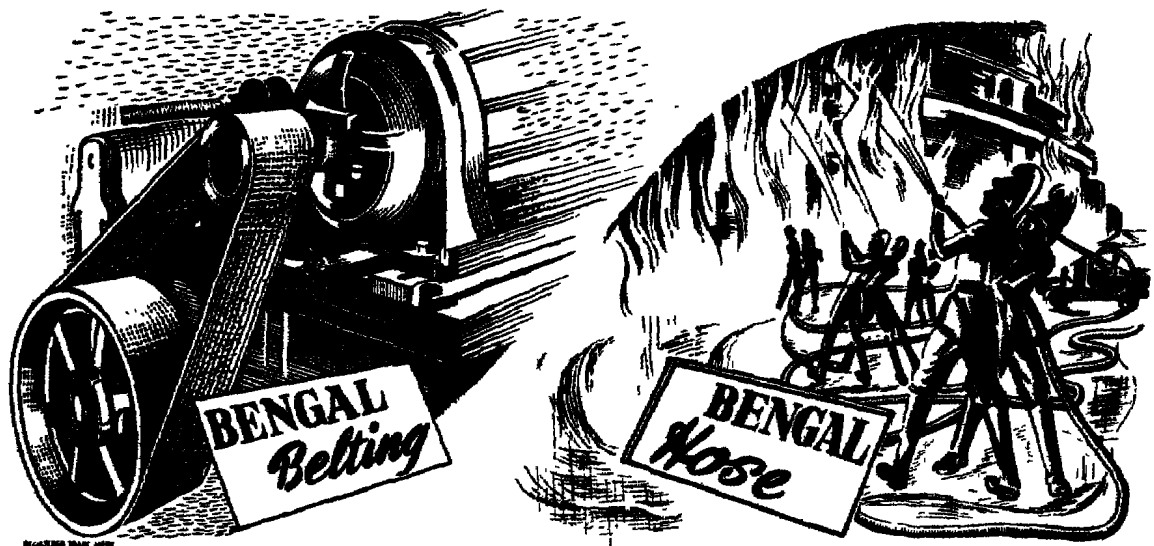
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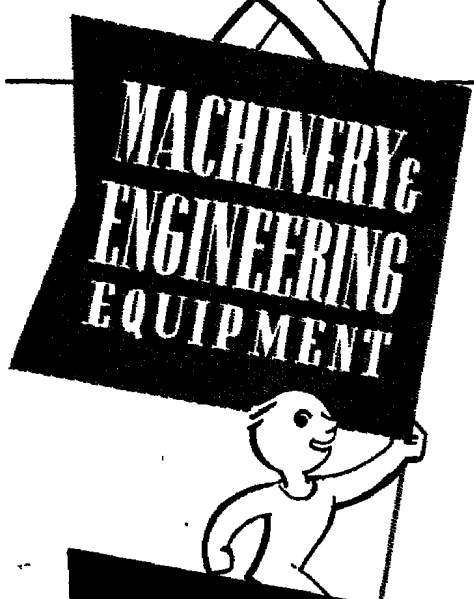
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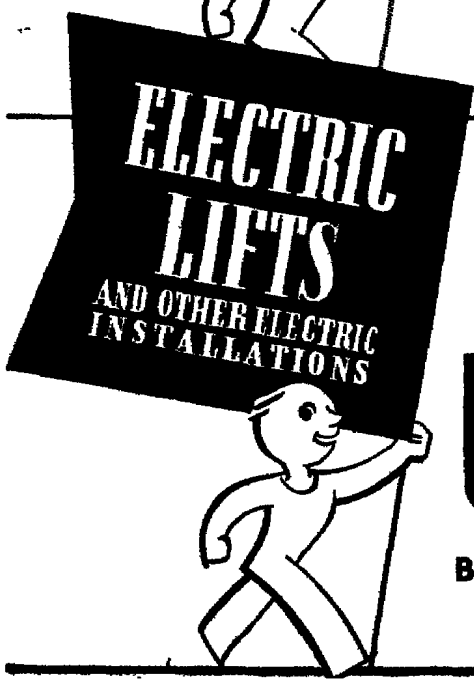
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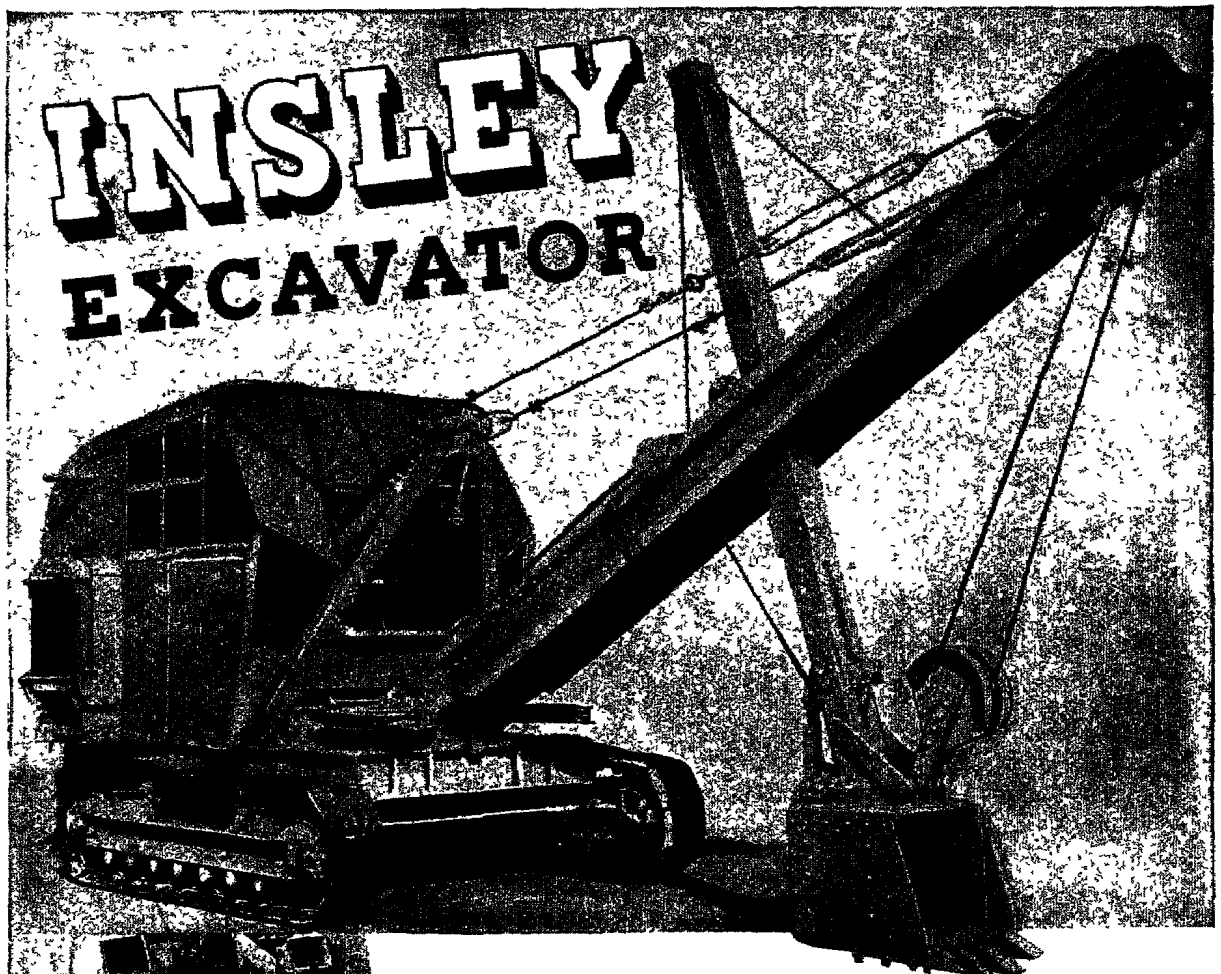
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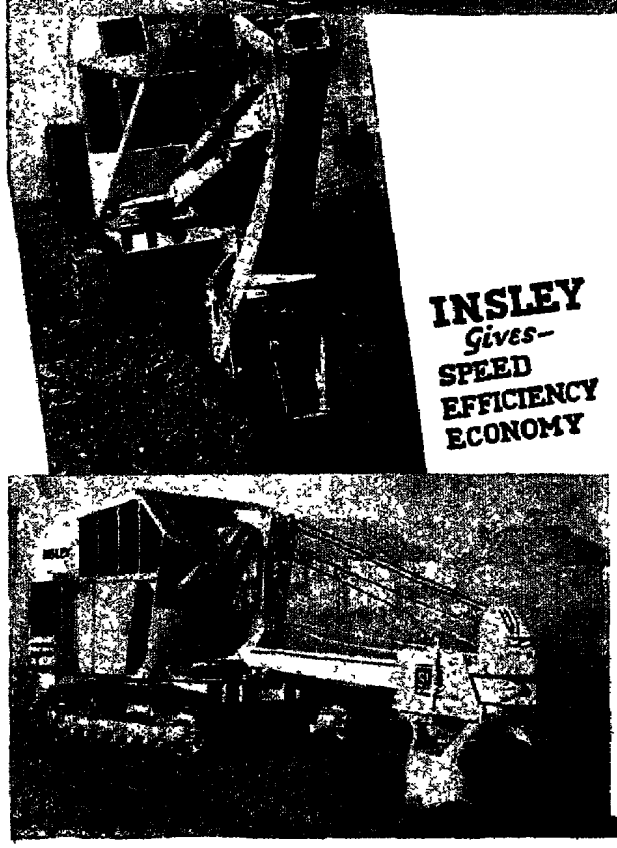
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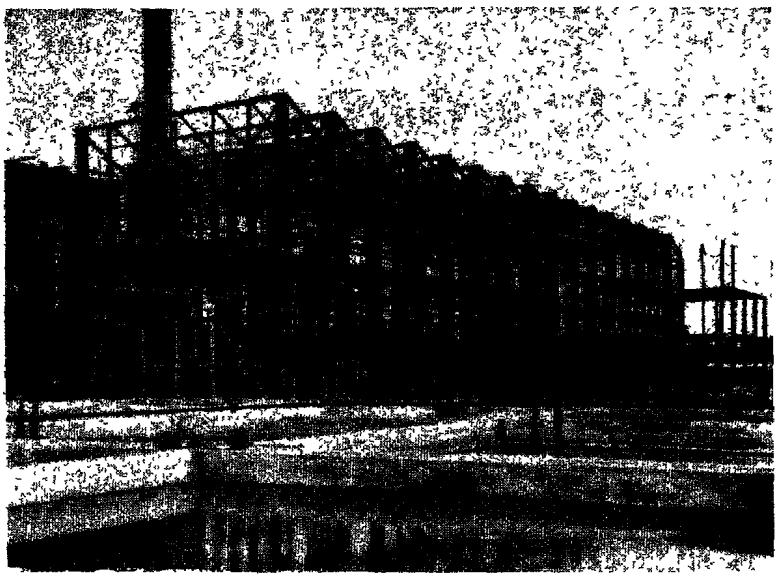
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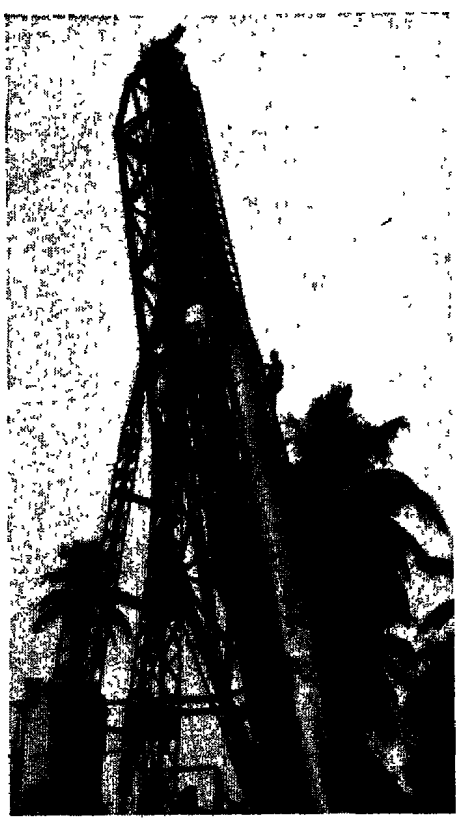
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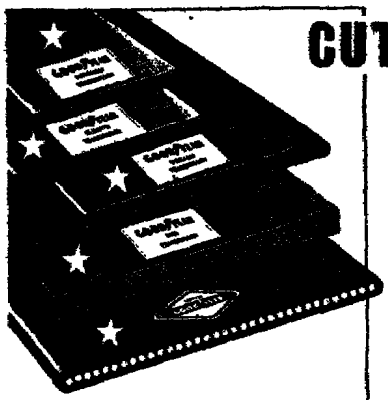


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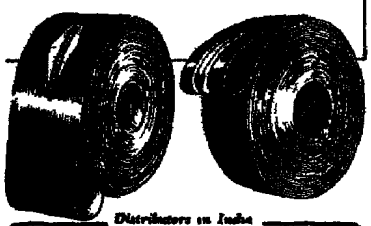
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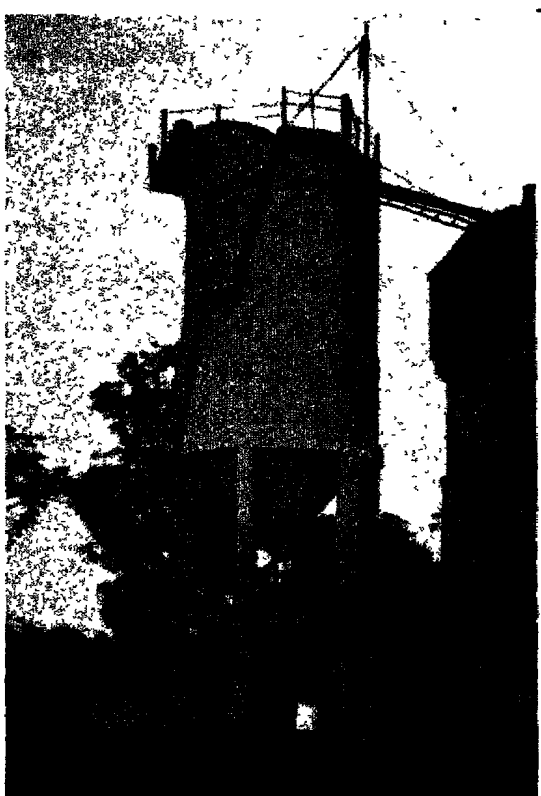
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December 15th, 1946

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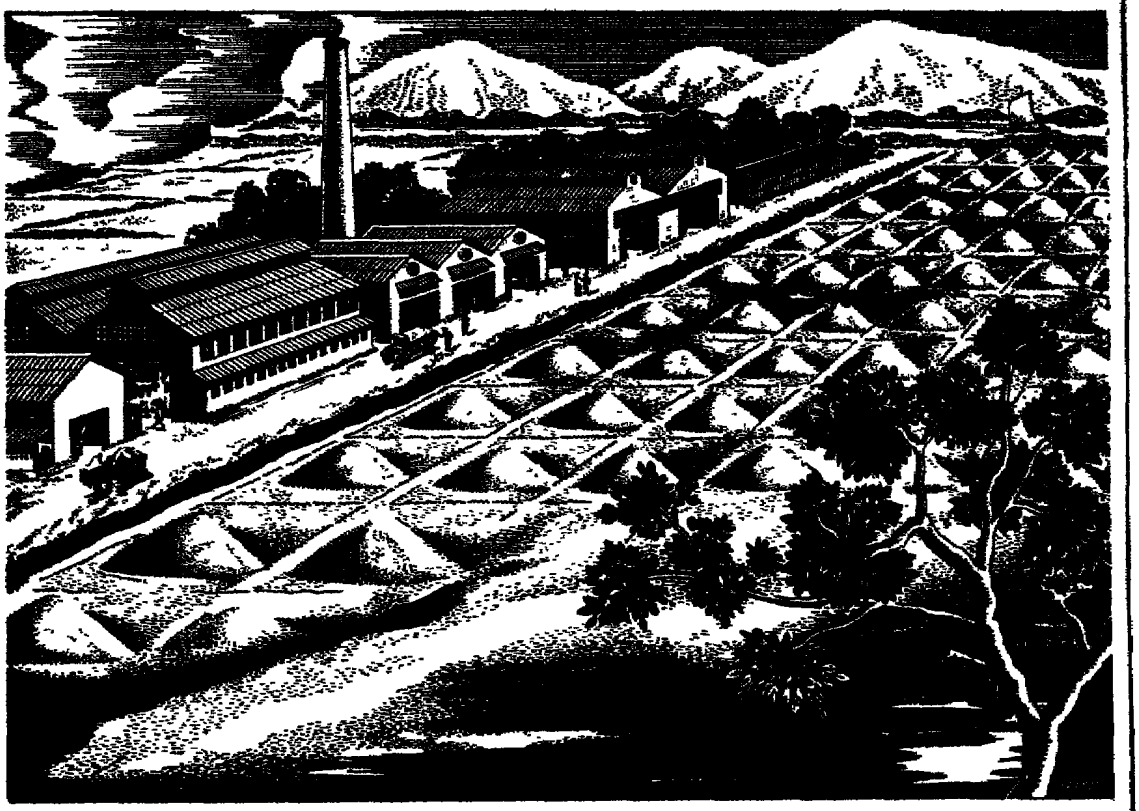
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EDITORIAL NEWS & NOTES

PLANT FOR PRODUCING PRE-CAST HOUSING PARTS

THE first foam slag pre-cast slab factory on straight flow lines in Britain contains interesting mechanical aspects designed specifically for this purpose. Built by the Glasgow Corporation Housing Department the factory is designed to produce wall sections for municipal housing. The slag is a cellular, lightweight aggregate produced by inflation and chilling of molten blast furnace slag at a temperature of approximately 2550° F.

The factory was first planned and approved in 1944 after exhaustive tests of foam slag unit sections in housing, under exposure tests and in experimental practice since 1938 when the Corporation Research Department first mooted the possibilities of foam slag. The factory is a single storey building with a second storey to the rear holding the mixing plant. It is in duplicate, and is built along a central line, the only unit shared by the two sides being the centrally located rear boiler plant which supplies heating to the curing ovens, etc. A description of one side implies a duplicate on the other.

Aggregate is delivered to storage bins located at the side of the plant, with a boom scraper, operating semi-circularly and feeding into a hopper from which a high bucket elevator carries the aggregate to the hoppers above the mixer.

There are two such hoppers feeding by gravity into the mixer hopper. Cement is fed in through the cement store, in the rear of the plant and delivered by cradle on an elevated runway to the cement hopper into which bags are tipped. A worm conveyer takes the cement to an elevator which carries it to a point above the mixer. Water is stored in a 10,000 gallons elevated tank in the rear of the plant.

All constituents for the mix are fed in to the mixer on feeder belts which are controlled to the speed of the main mixer belt. The mixer is a specially designed A.B.C. automatic belt controller, maximum capacity 30 cu. yd. per hr. The constituents feed in on a pre-arranged basis under control while all belts and speeds are co-ordinated to ensure absolute perfection of mix. Failure of any one constituent to feed in, would lead to the automatic stopping of the entire process. The mix pours into the ground floor where moulds are handled by a combination of ground roller conveyer tracks and overhead runways. These runways are located everywhere in the plant and are used to carry the heavy steel moulds and slabs through all the processes. The moulds are steel framed with perforated steel side-plates. Steel rod reinforcement, spot welded on the site in an adjacent welding shop is inserted in the mould and located partly by blocks and partly by hand while foam slag is poured in. The mould is then lifted by travelling cranes which co-ordinate with the overhead runways and carried into the curing ovens. These run centrally through the plant, four on either side. Dividing walls are of foam slag slabs, and doors are steel, steam tight and specially sealed. The moulds are fed in 30/40 at a time and cooked for 24 hours at a steam pressure of approximately 130 lb. per sq. in. The steam is carried in floor ducts down the centre of each oven, with outlet ducts at 4 ft. 6 in. centres while dry heat is provided by a bank of pipes down one side of each oven.

On completion of this process, the slabs are carried on the overhead rail system by travelling cranes, to the dismantling shop where the moulds are opened, the slab extracted and then taken for storage. The storage system used in the factory involves a series of steel racks, against which the slabs are clipped awaiting transport. A loading bay is so arranged as to permit easy handling of slabs from the storage bays. Special transportation has also been arranged still further to ease the problem of handling. Low trailers have been fitted with double steel inverted V shapes, crossed by wire reinforcement. These steel shapes correspond roughly to the steel racks and permit the loading of eight slabs to a trailer, in such an erect fashion as to permit easy lifting at the delivery end where similar mass erection methods operate. The slabs are grooved at the ends to allow the use of a foam slag mastik, which will grout the various units together. The two sections of the plant correspond precisely but one filling bay will be used mainly on straight wall units while the other filling bays will be retained largely for angle sections and other special items.

Outer wall sections are 6 in. thick and inner walls 4 in. Largest unit which the plant can produce is 10 ft. long by 8 ft. 8 in. high. The factory is perhaps the only one in existence in Britain and is certainly the most modern. In its construction the department met all sorts of problems which hampered the final use of the plant while a considerable number of appliances, equipment and components had to be specially designed to meet the particular needs of this plant.

The cost of the plant is not indicated nor has there been any indication to the hoped-for output per year. It was originally estimated that the plant would produce 2000 houses in 1946 but this figure is now very much subject to revision although it is still indicated that the 2000 is the maximum output.—(*"Mechanical World", Manchester*).

STABILISATION OF COMPLICATED JUNCTIONS BY GROUTING

Measures taken in America to stabilise main lines in open country by pressure cement grouting have been described in our columns from time to time. Generally similar measures, however, have also been adopted for complicated junction trackwork, notably in the 367-track-mile territory of the St. Louis Terminal Railroad Association. There, a number of groups of from three to twelve crossings, involving double slips and special installations, formerly required abnormally heavy maintenance due in the main to their being on embankments over soft blue clay sometimes 60 ft. in depth. Among them is Grand Crossing, where three tracks cross three others on the level almost at right angles, and Relay Station with twelve crossings carrying some 700 traffic movements daily. Various mixtures of cement, sand, and water were tried but considerable difficulty was experienced with plugged discharge pipe lines, and eventually neat cement and water was adopted as standard, in the proportion of 6 to 10 gal. of water to each sack of cement. Up to 1,000 gal. of such grout have been injected in an eight-hour day, and, apart from a slight increase in the batter and wear on certain crossings—due to the reduced cushioning effect of the consolidated ballast and subsoil—results are considered to be highly satisfactory. Maintenance costs have been reduced by more than 50 per cent.—(*"Railway Gazette" London.*)

* * * * *

MODERN EQUIPMENT FOR MATERIAL TESTING

Messrs. Alfred J. Amsler & Co. of Schaffhouse, Switzerland, well-known makers of testing machines for all materials, are sending an expert to India to advise manufacturers, industrialists, scientific institutions, etc., on the most modern methods and equipment for material testing. This equipment does, of course, include machines for the testing of cement and concrete, and a number of their machines are already in use in India. This expert will be at the disposal of organisations who would like to avail themselves of his advice. He is expected to arrive in India by the middle of December and will tour the whole country. Communications should be sent to: Volkart Brothers, Engineering Department, P.O. Box 199, Bombay.

We wish our readers A Merry Xmas and A Happy & Prosperous New Year

REINFORCED CONCRETE FLOATING DOCKS AT COCANADA

(Contributed)

THE "factory" for producing floating docks is an expensive assemblage of plant and equipment collected round a graving dock, locally called a "basin". Except in war time, many years of steady output of docks, ships, barges or other similar products would be required to justify the large capital expenditure involved in building and equipping the graving dock.

The concrete slabs are made alongside the basin on the casting area. They are lifted by cranes and placed in position in No. 1 berth in the basin. Decks, joints and spine wall are concreted in situ and the dock floated forward to No. 2 berth. Here machinery and fittings are installed and the top deck concreted When the next dock in No. 1 berth is ready to move forward, the dock in No. 2 is floated out and moored at the fitting out berth. Fitting-out and painting is completed and the finished dock towed out to deep water for testing When the "factory" is in full production work is in progress in each basin on 4 docks at a time. One is being finished off in the fitting-out berth, another is in No. 2 berth, the third is being assembled in No. 1 berth and slabs are being cast for the fourth in the casting yard An equal time is allotted to all four stages and a complete dock is turned out every four weeks.

GRAVING DOCKS

Reasons Governing Choice of Site

For the construction of the Reinforced Concrete Floating Docks, which were 264′ long × 66′ wide, it was necessary to have a large area of flat ground adjacent to the sea or to a canal leading to the sea wherein the tides gave a sufficient rise to allow the docks to be floated out of the basin and taken to sea.

Such a piece of land was located on the South side of the canal at Cocanada, this land being in use as salt flats

Cocanada District was also considered suitable as very little Government construction work was proposed and a fair amount of unskilled labour could be readily obtained. In addition, a branch railway line connected the main Madras-Calcutta line at Samalkot to the canal bank, directly opposite the proposed site, and materials could be transferred from this railhead across the bridge over the canal, a total distance of approximately one mile, or across the canal in barges direct to site.

Test holes were driven and the soil was found to be a type 'B' black plastic clay similar to black cotton soils of Australia. The sides of the test holes when cut stood vertical for several weeks and, provided care was taken to distribute the load evenly on the surface, the ground was considered satisfactory.

The 'Basin' or Graving Dock necessary for the construction of Reinforced Concrete Floating Docks, had to be 600′ long × 74′ wide, the bottom or floor level being at—5′ in the first 300′ and —6′ in the second 300′. To ensure that the site would not be flooded at exceptionally high tides, it was decided to fill in the working area to a level of +8′ and, as records showed that the tide had on one occasion reached a height of +8′, the surrounding bunds were increased to + 8.5′.

600 lineal feet of basin was required so that the basin could be utilised for the construction of two docks, the first 300′ (Stage I Berth) being used for the partial construction of a dock to a condition in which the basin could be flooded and No. 1 Dock floated down the basin into the second 300′ (Stage II Berth). The basin was then dewatered, No. 1 Dock settling down on its pebble bed in Stage II Berth.

Work of construction of No. 2 Dock was then commenced, in No. I Berth while additional work was carried out on No. I Dock in No. 2 Berth.

When No. 2 Dock was in a condition to be floated down to No. II Berth, the basin was flooded, gates removed; No. 1 Dock would then be taken into the canal, No. 2 Dock floated into No. II Berth, gates refixed and the basin dewatered.

No. 1 Dock would be taken to a 'Fitting Out Berth' built alongside the canal bank.

In order to accommodate the partially completed dock, three dolphins were built from the canal bank into the canal and a 5 ton Scotch Derrick Crane erected on the canal bank, the canal having been dredged at this point to—10′.

In this manner it would be possible for work to go forward simultaneously on three docks, while precast slabs are being prepared for a fourth.

Owing to the success of the Reinforced Concrete Docks in various theatres of the war, it was decided that the project had to be duplicated immediately in the vicinity of the first basin so that the output of Docks could be increased, and work was accordingly commenced on a second basin. This basin was almost completed when peace was declared and the further construction of Reinforced Concrete Floating Docks was abandoned.

Method of Construction

Basin construction consisted of driving 27′ long Frodingham steel sheet piles along the sides and ends of the Basin, the top of piles being left at +8′ level. These piles were tied back in the usual way using tie rods connected to reinforced concrete anchor beams running parallel to the sect piling but 35′ outside it. (*See Fig.* 1).

The earth inside this steel box was excavated by coolies to a level of—6, 5′ on the first 300′ of the basin and—7.5′ on the second 300′. 9″ of laterite boulder soling was then laid on the floor of the basin. The first 300′ of the basin floor was finished off with a 9″ reinforced concrete floor having a 1″ timber boarded floor above the concrete with a 1″ gap between and approximately ½″ spacing between the floor boards. This was necessary to allow water to flow underneath the concrete dock when finished to give the necessary flotation. The second 300′ had a floor of rounded pebbles, approximately 9″ deep, providing a loose surface on which the newly constructed dock could bed down evenly. The bund between the basin and canal was left intact until all work in the basin had been completed and the dock gates, which were of the sloping bear-trap type in 4 leaves, were completed, together with a sump 30′ × 9′ at the corner of the basin nearest the canal.

The dock gates were constructed in 4 leaves, the framework being steel sections with a caulked timber surface fitted with sluices, and could be lifted out and replaced when necessary by American Stiff-Leg Derrick Cranes.

With the development of the various types of landing craft came the need for a means of docking them, particularly the larger types which could not easily be hauled up on slipways. Floating docks which could be towed to wherever the need arose were the obvious answer. These had previously been built of steel but shipyards in U.K. were all working at full capacity. Mr. G. A. Maunsell then developed his idea of a Concrete Floating Dock which could be built in a graving basin using only the most easily rolled form of steel and capable of being built by a section of the construction industry not at that time so heavily engaged on war work.

The docks are composed of a number of precast slabs held together by bottom, deck and vertical joints of concrete cast in situ. This was work which could be done by concrete contractors and relieve the already overburdened shipyards. Morever a concrete dock could be built more quickly than a steel one. Construction began about September 1943 and concrete floating docks have been used in England, in the Mediterranean and on the coast of France. The longest tow that any dock has made was from Gravesend to Rangoon in 1945.

The accompanying article describes the construction and equipment of Graving Docks and 800-Tonner Reinforced Concrete Floating Docks at Cocanada on the East Coast, some 100 miles south of Vizagapatam.

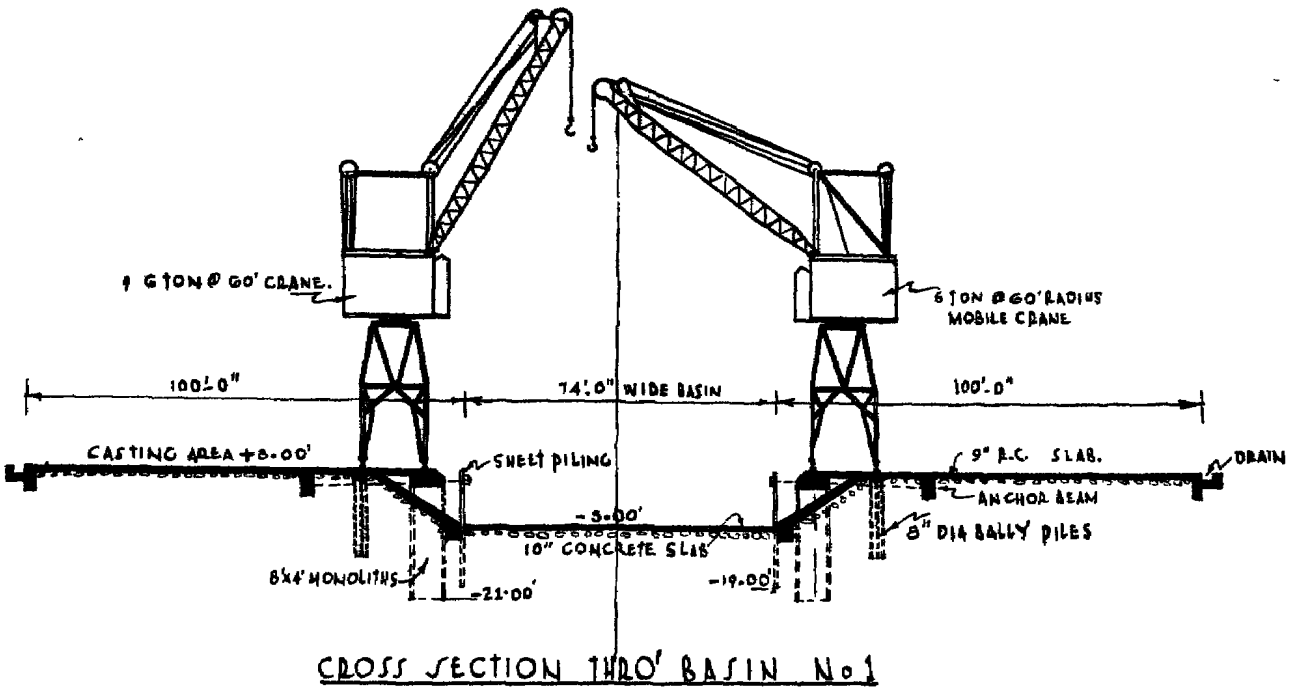


Fig 1 The Graving Dock locally called a "Basin" is 600-ft. long by 74-ft wide and utilised for the partial construction of two Docks, at two different stages of preparation

A Pump House containing four diesel driven pumps to give a capacity of approximately 4,000 gallons per minute was built alongside the sump for dewatering the basin

Casting Yard

Along each side of the basin is a concrete slab 100 feet wide A width of 30 feet from the dock wall is occupied by the crane tracks, leaving 70 feet for the working space on which to produce precast slabs for the floating docks. (*See Fig* 1).

The working area along each side of the basin was built up by excavations from the basin and from mud dredged from the canal, to a height of +6 5', the whole area being soled with laterite boulders for a depth of 1', rolled with a 6 ton roller and a 6" reinforced concrete slab constructed above the soling

The only cranes available for this work were Stothert & Pitt 6 ton Diesel Driven Electric Cranes which gave a point load of approx 47 tons when in use, and crane tracks had to be built along each side of the basin to support this crane load To support this track 8" ballies 18' long were driven in sets of 3 across the track 3' apart, every 3' A strongly reinforced concrete beam approximately 3'-0" × 2'-6" deep was constructed on the top of these piles, each reinforced beam carrying one rail

Work of partially excavating the basin was commenced early in December 1944 and driving of sheet piling commenced towards the end of December.

At the beginning of February 1945 it was discovered that the tops of the piles were being pushed steadily inwards to the basin at a rate varying from ¼" to ½" per day and, in spite of various methods being used to stop this creeping of piles, the inward creeping continued and the whole working area, including anchor beams, reinforced concrete slab, crane track and additional anchor beam at the outer edge of the slab steadily moved inwards carrying the piles with them From observations it appeared that this was caused by a head of water building up behind the piles flowing through horizontal stratas of sand which existed at various levels in the ground. It was finally decided to excavate the earth outside the piles and to sink monoliths to support the inner crane beam for the first 300' of basin and to use travelling Scotch Derrick Cranes, which had been obtained by the M.E.S, for the second 300'. These monoliths were rectangular, 8' × 3' built up of prefabricated sections in reinforced concrete 6" thick and were sunk by weighting and internal excavation The monoliths were sunk to an average depth of —21', sealed with concrete and filled with wet sand. On actual tests these supported



Fig. 2. Casting Yard. On either side of the graving dock is the casting yard where prefabricated units are cast in stacks, the heaviest slab being about 6 tons. Note protruding reinforcement which is later incorporated in in-situ concrete.

a 60 ton load with little more than the initial sinkage of approximately ¼".

Sand and Aggregate

Sand was obtained from the Godavari River, approximately 45 to 50 miles from the site, and carried there by barge

After tests, it was decided that the best available aggregate for the construction of the docks was a black quartzite available at Rajahmundry about 40 miles away This could be brought by barge along the canal except when this is closed during the latter part of the dry weather. Then material must come in by rail and aggregate taken to the crushers by truck

Sand and aggregate are distributed to the various mixers by 10 No 2 Cu Yd. Dumpers

For the thicker slabs the maximum size of coarse aggregate is ¾" but for most of the work it is broken to ⅜" Primary crushing is done by jaw crushers set up on staging with a section of ¼" mesh screen in the delivery chute to eliminate some of the dust From the crushers it is carried by head load to roll granulators, dust and oversize material screened off, and stock piled

Mixing Plant

Each basin has eight half-yard mixers set up in pairs on each side of each berth With the mixers at, or near, the mid points of the berths concrete can be transported to slabs by head loads For placing in situ concrete, skips, handled by the cranes, are used Aggregate is brought in by dumper and a small stockpile maintained behind each mixer At each mixer site is a Nissen hut for cement This decentralisation of mixers is preferable to a central plant when coolie labour is available as it eliminates one link in the transport chain With a central mixing plant decauville or some such means would be required to deliver the concrete near where it is to be placed but the final move would remain the same, head loads to slabs and skips lifted by the cranes for in situ work in the basin In addition to the eight main mixers there are to be two mobile machines at each basin for miscellaneous small jobs

FLOATING DOCKS

General Description

The docks are cellular reinforced concrete structures designed on geodetic principles The walls, the bulkheads and the gallery deck are of thin precast slabs Not only does this precasting help to speed construction but it also minimises shrinkage cracks formed during the curing of the concrete The bottom, the spine wall, the pontoon deck and the top deck are cast in situ, as are also the vertical joints between slabs and four small in situ walls at the swim ends To completely tie in all slabs, reinforcement is left projecting and is bent as required to mesh in with adjacent slabs and the in situ concrete Timber blocks are fixed to the pontoon deck to take the weight of docked craft and timber fenders are fitted to the sides. The docks are equipped with diesel driven generating sets, electric welders, pumps and with crew's quarters.

TABLE No. 1.

1.	Lifting capacity, Tons.	300	800	1,000
2	Length	210′ 0″	264′ 0″	267′ 0″
3	Beam	61′ 0″	64′ 0″	67′ 0″
4.	Height of side	28′ 0″	24′ 0″	32′ 0″
5	Draught	6′ 10″	6′ 8″	8′ 2″
6	Approx. Displacement, Tons		2,850	3,900
7	Width of Pontoon Deck	40′ 0″	49′ 0″	48′ 0″
8.	Width of each top deck	7′ 6″	7′ 6″	9′ 6″
9.	Height of pontoon deck above bottom	8′ 9″	8′ 10″	10′ 6″
10	Thickness of bottom slab		5″	5″
11.	Thickness of pontoon deck		4″	4½″
12.	Thickness of top deck		7″ to 13½″	7″ to 16½″
13.	Volume of concrete, Cu. Yds		1,216	1,757
14.	Weight of Steel, Tons		382	538
15.	No of main slabs (No 1 berth)		174	330
16	No of Gallery slabs (No. 2 berth)		36	44
17	Total No of slabs		210	374

N B —Fig 4 shows the outline of an 800-ton dock and a cross section through the pump room

Dimensions and Quantities

There are three sizes of docks to be made in India with lifting capacities of 300, 800 and 1,000 tons Details are given in Table 1, full details of the 300 tonners not yet being available

The cubic yards of concrete going into the various parts of the docks and the general order in which they are cast are given in Table 2

TABLE No 2.

	Size of Dock Tons	800	1000
		Concrete Cub Yds	
1	Slabs	565	875
2	Bottom	230	260
3	Joints	81	187
4	Spine Wall	52	70
5	In situ Walls	12	29
6.	Pontoon Deck	100	209
7	Top Deck	110	127
	Totals	1,216	1,757

Item 1, of course, is precast Items 2, 3, 4, 5 go on more or less simultaneously, with item 2 leading, in No. 1 berth. Item 6 is cast in No 1 berth for 800 tonners but for 1,000 tonners will probably be done in No 2 berth. Item 7 is poured in No 2 berth after placing gallery slabs and machinery.

Precast Slabs

There are ten main types of slabs and each type contains several variants in reinforcing, apertures, left and right hand slabs, etc Lifting eyes are cast into all slabs As with all the rest of the dock, the slabs are very heavily reinforced and the steel is left projecting round the edges 18" × 24" to mesh with the steel of adjoining slabs in the in situ concrete of the joints This introduces difficulties in edge shuttering, lifting and placing The maximum weight of slab is about 6 tons. Table 3 gives details of the types of slabs that go to make up an 800-ton dock For other sizes of dock the slab types are similar

All slabs except Type J from the vertical sides and bulkheads of the dock and are placed in No 1 berth The gallery slabs, Type J, from the only precast deck and are placed in No. 2 berth

Slabs have projecting reinforcement for incorporation in the in situ concrete of decks and joints. As the slabs are cast in a horizontal position and placed in a vertical one turning presents difficulties Lifting lugs are provided in all slabs but the method of handling depends

TABLE No. 3

	Type	Position.	Thickness.	No. of variants.	Total No. of slabs.
A	Outside slabs	Outside walls	5″	6	36
B	Bridge slabs	Inside walls	4″	8	40
C	Small end slabs	Dock walls	4″	6	5
D	Swim ends	Ends	4½″	2	4
E	Small triangles	Ends	4″	2	4
F	Swim ends	Bulkhead walls	7″	2	4
G	L Slabs	Cross members	4″	3	60
H	L Slabs	Bulkheads	4″	2	11
I	L Specials	Engine Rooms	4″	4	7
J	Gallery Slabs	Gallery deck	2″	10	36
		Totals.		45	210

on the type Some have sufficient beam strength to be lifted from one end directly into the vertical position. These slabs are cast along trenches provided with thin cross walls on which the slab rotates during lifting, while the projecting reinforcement goes down into the trench. Other slabs are too large to stand this treatment and have to be turned on tilting tables There is one of these on each side of a basin and they consist of a frame work of steel channels to take the slab The frame is supported on short columns so as to rotate from a horizontal to a vertical position and inclined props drop into recesses to hold it there The slabs are placed on the table horizontally, the table tilted and the slab picked up again in vertical plane Many slabs require strengthening before lifting Some have bolts cast into them for the attachment of strong backs while on other types the strong backs are held by hooks engaging in the reinforcement

The casting of prefabricated reinforced concrete slabs to be used in actual dock construction was commenced on the casting area on 4th March 1945, and the laying of reinforcing steel for the bottom of the dock was commenced on 30th April 1945

In Situ Work

Unlike the slabs, which consist of a number of small individual pours, none more than about 3 cubic yards, this is placed in as large sections as possible, usually three, with transverse, square edged construction joints The spine wall between the bottom and the pontoon deck is poured in sections and then the pontoon deck Work on the top deck is complicated by very heavy reinforcement closely spaced and by the No of openings for hatchways, etc Concreting of vertical joints is similar to work on columns in buildings and they are all small pours Owing to the quantity of steel in them, approx. twice that in the slabs owing to lapping of rods there, concreting is difficult and a wet mix is essential The large amount of steel helps to control the shrinkage cracks which might occur due to the sloppy mix

Plant & Accommodation

Each dock is equipped with two 2,600 gallons per minute pumps for dewatering which can be used either singly or together. There are two generating sets and two separately driven welding sets All plant is diesel driven Fuel and fresh water tanks are installed Sleeping, messing, cooking, washing and lavatory accommodation is provided for the dockmaster and crew of 16. These quarters are required for the crew which must be provided when the docks are being towed to the ports in which they are to be used

FLOATING DOCK CONSTRUCTION

In the following paragraphs the building of an 800 ton floating dock is described. Construction of other sizes follows the same general principles.

Concrete

The specified mix is 1 . 1 . 2 . 2.4 and the minimum strength required is 2,500 lbs per sq in at 7 days and 4,000 lbs sq in at 28 days With such a rich mix these strengths should have been easily exceeded but actually preliminary tests barely achieved them due to dust and dirt in the aggregate Water for washing is not available but by more thorough screening off of crusher dust and more care in storage and transport of aggregate the 7 day strength was raised to between 3,500 and 4,000 lbs per sq in The richness of the mix may seem extravagant but the cost of the extra cement is a small premium to pay for assured water tightness and to cover poor workmanship The large amount of cement would introduce some shrinkage trouble were the docks cast as monoliths but precasting and the relatively thin, heavily reinforced, in situ work minimises this danger Hot weather, however, did cause complications Temperatures did not exceed 108° in the shade but the steel got very hot in the sun and had to be cooled before pouring concrete This was done by covering the fabricated reinforcement with chittai and keeping it wet. Keeping slabs wet while curing is done in the same manner and sometimes mud bunds are built round the edges and the surface flooded Slabs of the same type are cast in stacks one on top of another there being up to 7 slabs in a stack At times only 24 hrs elapses after the casting of a slab before another is cast on top of it.

An unusual aspect is the importance of the weight of the concrete A heavy concrete increases the displacement and reduces the lifting capacity. The stone used at Cocanada is a trap rock with a specific gravity of 2 8 to 2 9 giving a concrete which weighs 154 to 160 lbs per cub ft without reinforcement This very heavy concrete would reduce the lifting capacity of the dock by some 150 to 200 tons A search was made for a lighter aggregate but only a weathered sandstone, specific gravity 2 55 to 2 6, could be found and that 100 miles away This produced a poor concrete with a 7 day strength of only 2,000 lbs. The heavier stone was therefore used The No of 800 ton docks to be made was reduced to three already under construction (2) and 1,000 tonners built instead In the 1 000 ton docks the design allowed for the heavier aggregate.

Steel

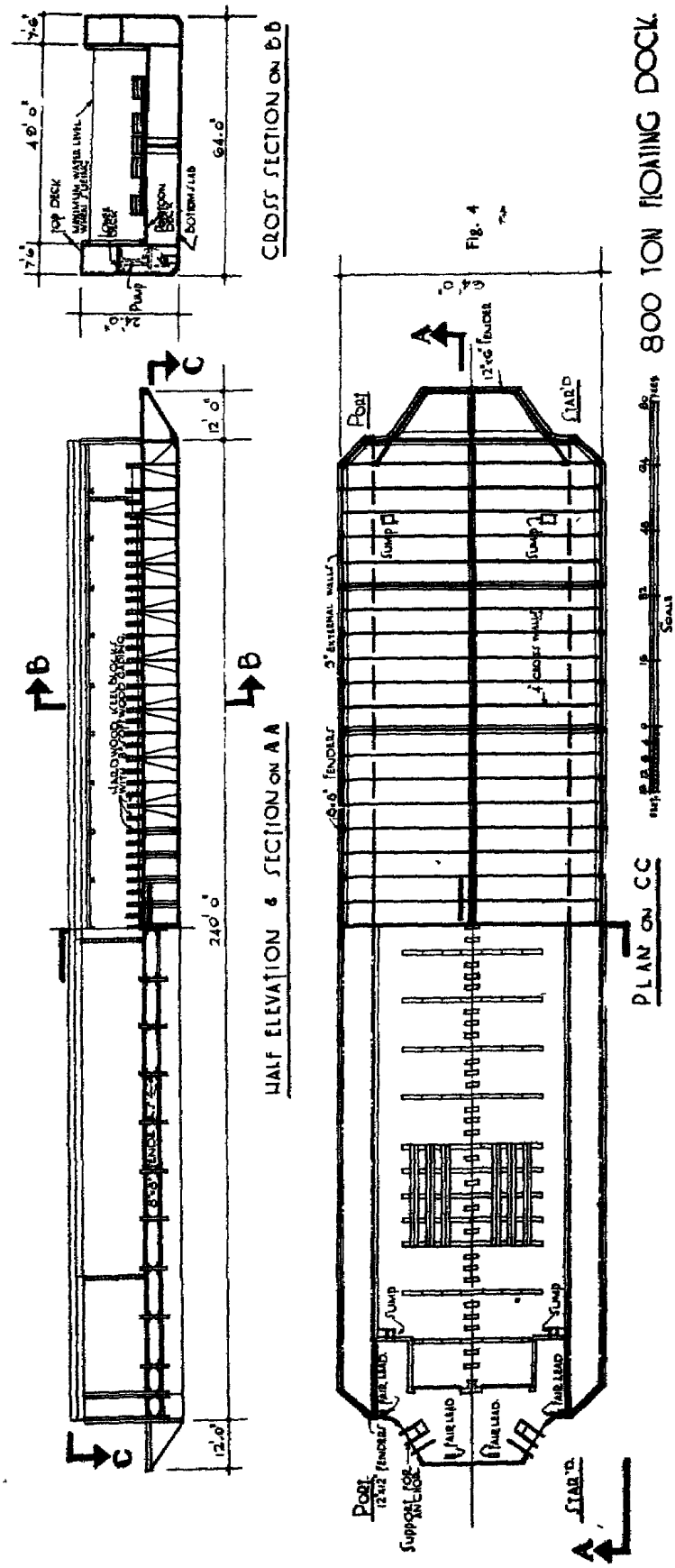
There is no complicated bending of steel but, with the average percentage of steel in the neighbourhood of 5 3 and considerably more in some parts, care has to be taken to place it accurately In the heavily reinforced members, like the top deck, there is little room for lapping and long lengths have to be used Similarly there is no room for hooking the ends of rods Cutting is done in the steel yards with bar croppers and electrically driven cutting machines Bending is done by hand for the smaller diameters and by power benders for the larger sizes This is all repetition work and stacks of the various types of bars are built up and drawn on as required When fabricating the reinforcement, rods at the lifting points are welded to ensure distribution of the load, and the remaining bar crossings are tied

Precast Work

The layout of the casting yard requires careful planning so that slabs can be moved from where they have been cast to their position in the dock with the minimum effort Tilting tables and pits for the reinforcement have to be sited near the slabs which require them Slabs of the same type are cast one on top of another with paper joints between them This means that it is always the youngest slab that is lifted first and the lifting stresses are the most severe that the concrete will receive Being cast on the flat, only edge shuttering is required but this has to be bored to pass the projecting reinforcement In all the main slabs there are two layers of steel, and edge shuttering is manufactured by boring 1¼" holes in a piece of



Fig. 3. Dock in Erection The precast units are lifted by crane, assembled in the basin and tied in by in-situ concrete. Precasting helps speed up construction and also reduces risk of shrinkage cracks.



timber, ripping through the centre line of the holes, then placing one piece with the half holes uppermost to fix the position of the lower layer of reinforcement in relation to the bottom of the slab. A strip of timber as thick as the distance between the two layers of steel is then nailed on over the bottom steel. The top steel is placed and then the other half of the bored timber is nailed on top, its flat edge forming the screed guide for the top of the slab

Work in No. 1 Berth

The first work is the building of the chamfered edge shuttering for the bottom slab and the erection of the supports for the sloping swim ends. The timber floor of the berth is then covered with paper and the bottom reinforcement placed Then begins the placing of the precast slabs The slabs are lifted, the projecting reinforcement bent as required for the particular joint formed at the bottom, and then lowered into position in the basin. The first slabs placed are the cross members and the bulk heads, the L slabs. The first of these is strutted in position by an A frame engaging in the top of the slab and subsequent ones are held upright by pairs of timber spacers. These are timbers of the correct length with two cleats at each end which engage on each side of the top of the slab. Final adjustment of position is made with fix wedges. In the ends of the L slabs are screwed rods which pass through holes in the inside and outside slabs nuts are screwed on and the whole assembly then forms a self-supporting structure. The side slabs cover two bays of L slabs so that the sides are held by an L slab at the centre and form a common vertical joint with the L slabs at each end. Swim end slabs are placed against their supports while the gallery slabs are not placed until the dock is in No. 2 Berth.

When the slabs are in position and the projection steel in the joints adjusted, the bottom is concreted, the work usually being done in 3 pours. Shuttering is then put up for the joints and for the in situ walls. As these are concreted, they are all small pours, shuttering for the spine wall is erected, reinforcement placed and the wall cast. Shuttering is then struck and used for the underside of the pontoon deck. In the case of 800 tonners this must be cast in No. 1 berth but in the 1,000 tonners it will probably be left until floating forward to No. 2 berth.

The basin is then flooded, the dock in No. 2 berth floated out into the canal and that in No. 1 berth moved to No. 2. Slabs for the next dock having been cast while the one that has just quitted No 1 was being assembled, work on the No 1 berth stage of the following dock begins immediately. The work to be done in this berth controls the whole sequence of operations as the dock in No. 2 cannot float out till that in No. 1 is ready. Much of the work done in No. 2 could equally well be done at the fitting out berth.

Work in No. 2 Berth

In this berth the dock beds itself down evenly on the loose pebble bed so that there are no stress-concentrations due to

projections on the surface of the young concrete of the bottom of the dock. The first work done is the placing of the gallery slabs and the preparation of the beds for the machinery Generators, welding sets and pumps are placed in position and the shuttering assembled for the top deck. Machinery can be placed through hatches but in the first instance it is very much easier to do the work before the deck is cast. Longitudinal steel is very crowded in the top deck There is approx the same cross sectional area of steel as in the bottom, but there is only 25% of the width and even this is still further reduced by hatchways and other openings The thickness of the top deck of 800 tonners varies from 7" at the ends to 8½" and then 13½" over the central portion where, in addition to higher stresses, the reinforcement is most crowded due to hatchways, and more layers of steel are required In the case of 1,000 tonners the deck thickness from 7" to 8½" and then, on the outsides of the hatch openings only 16½" making an L shaped slab. At the same time all external walls of the dock are given two coats of plastic bituminous paint and then the fenders fixed except those at the corners which go on in No 1 berth The crews' quarters are fitted up, pipes, valves, tanks and electric wiring installed and timber keel blocks fixed on the pontoon dock

Work in Fitting Out Berth

Essential work in No 2 berth is placing of heavy machinery, concreting painting of exterior walls and fixing of fenders As much of the remaining work as possible is done in No 2 berth and then, when floated out, the dock is warped to the fitting out berth The remaining work in the crews' quarters and fixing of capstans, handrails, etc, is completed there and walls of living and working accommodation painted with anti-condensation paint Exposed steel work is painted with black bitumastic and the inner and other external walls with camouflage paint Provided there is no delay in delivery of fittings all this work is finished before the next dock is floated out but it is essential that a good start be made with fitting out while the dock is in No. 2 berth.

Testing

When completed the dock is towed to deep water and sunk to its maximum depth, leaving two feet of free board to the top dock It is maintained in this position for at least one hour and observations made by sounding the tanks, etc., for any possible leakage. In docks which have developed leaks the seepage usually occurs at the blocks on which the L slabs are placed before the bottom is concreted. To counteract this the blocks are rendered with Sika compo before the slabs are lowered on them. Before floating forward from No. 1 berth the bottom is dried off and then examined for seepage when the basin is flooded. Leaks are rendered with Sika After the test sink 6" of water remains in the bottom of the dock and no further visual examination for leaks there can be made. Some docks that have met rough weather on long tows have developed slight leaks in the vertical joints, usually well above the normal waterline These have been cured by rendering with a water-proofing compo.



Reinforced Concrete Floating Dock for the Royal Navy

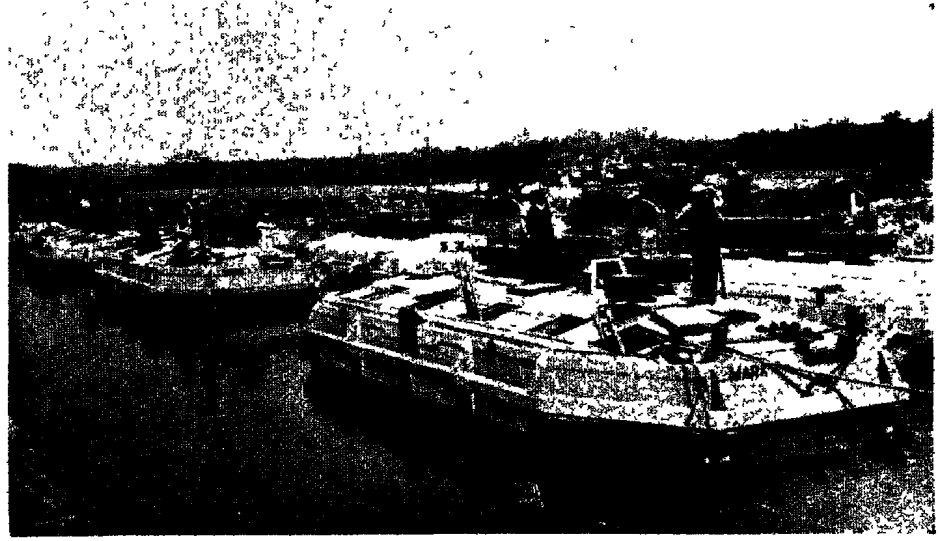
Built to replace a steel dock destroyed by enemy action, this reinforced concrete dock was launched from a site right next door to Bucklers Hard, from which were launched many of Nelson's men-of-war The dock did great work in lifting and servicing the ships of the Royal Navy.

Reinforced Concrete Pontoons for Mulberry Harbour

Mulberry Harbour's ten miles of floating pier were carried on reinforced concrete pontoons. They had to be light, but strong enough to carry the weight of the flexible steel spans plus heavy military traffic. Made from concrete panels only 1½" thick, every pontoon had over 1,400 feet of joint between precast and in-situ concrete, every inch of which had to be watertight.

Both the dock and the pontoons were constructed by Wates Ltd., a leading English firm of Contractors.

[Photos, courtesy: Wates Ltd.]



REINFORCED CONCRETE ROAD BRIDGE

By **W. SCOTT WILSON**

THE accompanying drawings indicate a method of spanning approximately 50 ft by means of a reinforced concrete girder bridge The carriageway is 20 ft between kerbs, and is designed to support 20 ton transports, 14 tons on the rear axle and 6 tons on the fore axle Two foot walks, each 5 ft 6 in. wide are provided for pedestrian traffic

The decking is composed of pre-cast concrete beams 24 in × 12 in in section, placed at 6 ft. centres upon the supporting main girders These beams are cast with ledges on their sides upon which pre-cast concrete segments, 5 ft long by 18 in wide by 2 in thick, take their bearing. After these segments are placed in position, 1-2-4 concrete is poured upon

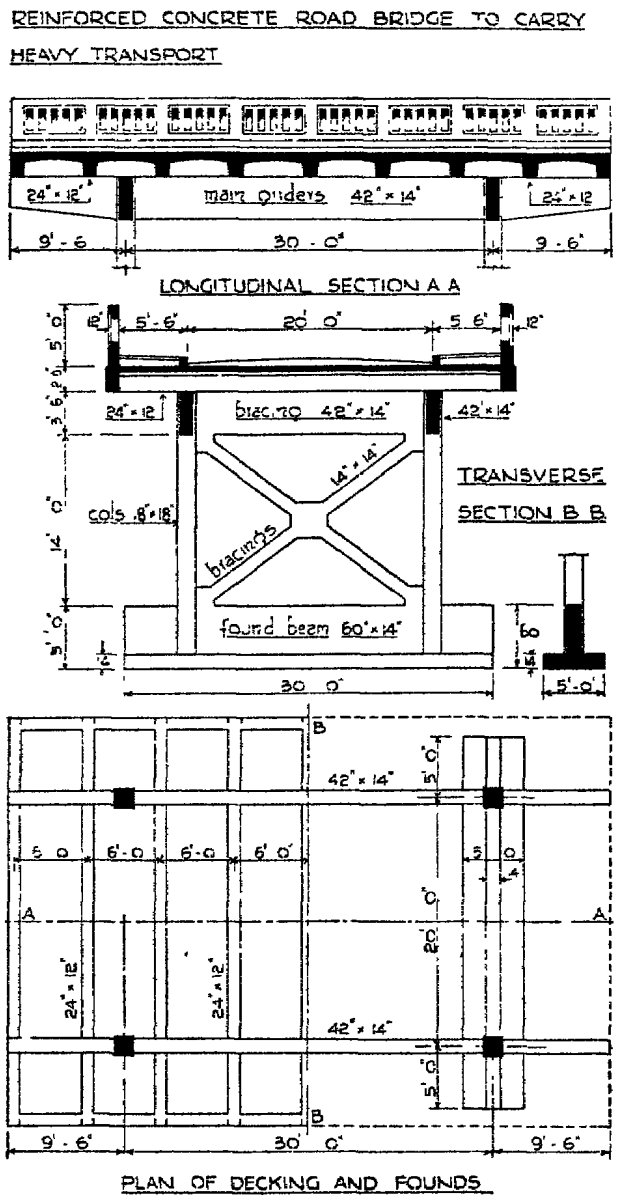


FIG 1.

them and screeded level with the tops of the beams, giving a depth of concrete of 6 in. at the crown and 9 in at the springings, as shown on the section, Fig 2 Any desired wearing surface can then be applied

By this means a strong decking may be obtained without the use of site timbering and props The foot walks are constructed with sand fill and 2-in pavement flags and the decking completed with concrete parapets as shown on Fig. 1.

The main girders, supporting columns, bracing and foundation slabs are all cast *in situ* and present no unusual features. It may be noted, however, that the cantilever ends of both main and secondary girders reduce the positive bending moments at mid-span and therefore have an economical effect on the size of beam and reinforcement required

Each foundation is 30 ft long by 5 ft. wide and supports a total load of 192 tons, and since the base area is 150 sq. ft. the ground pressure is approximately 1¼ tons per sq. ft.

Decking beams, section 24 in × 12 in, centre span 20 ft.

Dead Loads;

beam 24 in × 12 in	20×2·0×144	= 5,760 lb
concrete fill	20×5·0×84	= 8,400 lb
wearing surface.	20×5·0×30	= 3,000 lb.
total dead load for 100 sq ft.		= 17,160 lb
		= 8 tons
unit dead load for carriageway		= 170 lb per sq ft
unit dead load for foot walks		= 210 lb per sq ft.

Live Loads,

Two axle loads of 14 tons per beam = 28 tons
For main girders unit live load = 280 lb per sq. ft.

Cantilever Portion; section 24 in × 12 in; span 6 ft.

Dead Loads,

beam, concrete fill and foot walk .—
= 6·0×5 5×210 = 6,900 lb. = 3·0 tons
concrete parapet
=6 0 × 7·0 × 0 8 × 144 = 4,500 lb = 2·0 tons.
total dead load foot walks = 11,400 lb = 5·0 tons.

Live Load,

= 6·0 × 5·5 × 100 = 3,300 lb. = 1·5 ton
total dead plus live load = 14,700 lb = 6·5 tons.
dead load moment $M_1 = 3·0 \times 2·75$ = 8·25 ft tons.
" " " $M_2 = 2·0 \times 6·0$ = 12·0 " "
total dead load moment = $M_1 + M_2$ = 20·25 ft. tons
live load moment $M_3 = 1·5 \times 2·75$ = 4·13 " "
total max moment = $M_1 + M_2 + M_3$ = 24·38 ft tons.
steel area As = 24 4/8 × 1·5 = 2 sq. in.
total shear S = 14,700 lb.
unit shear s = 14,700/18 × 12 = 68 lb per sq in.
and no shear steel is necessary.

Centre 20 ft portion.

dead load free moment.
$M_4 = 8 \times 20/8$ = 20 ft. tons.
live load free moment
$M_5 = (14 \times 10) - (7 \times 7·5) - (7 \times 1·5)$ = 77 ft tons.
dead plus live load free moment = 97 ft. tons
deduct dead load cantilever moment = 20 25 " "
max positive moment at mid-span = 76·75 ft tons.
steel area As = 77/8 × 1·5 = 6·4 sq. in
and 8 bars 1 in diam = 6·3 sq in.
max shear due to dead load:
$S_1 = 8/2$ = 4·0 tons
max shear due to live load.
S_2 = 14·0 tons,
and total shear S = 4 + 14 = 18 tons.
unit shear s = 18×2240/18 × 12 = 187 lb. per sq. in.

Adopting the steel arrangement shown on Fig 2 :—
2 diagonals 1 in diam. carry:
1·57 × 8 × 18/40 = 11·3 tons.
4 sections ⅜ in. diam stirrups at 6 in. crs. carry:
0·44 × 8 × 18/6 = 10 56 tons
and shear resistance provided = 21·86 tons.
which is amply sufficient.

General details of bridge decking and supporting beams are shown on Fig. 2.

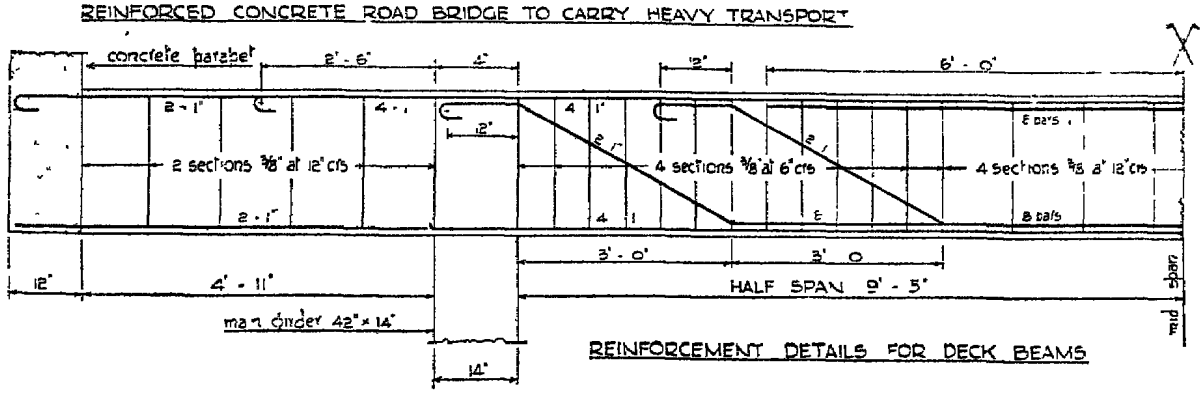
Main girders ; section 42 in. × 14 in cantilever span 9 ft 6 in

Total dead load from each deck beam
= 8/2 + 5·0 = 9 0 tons

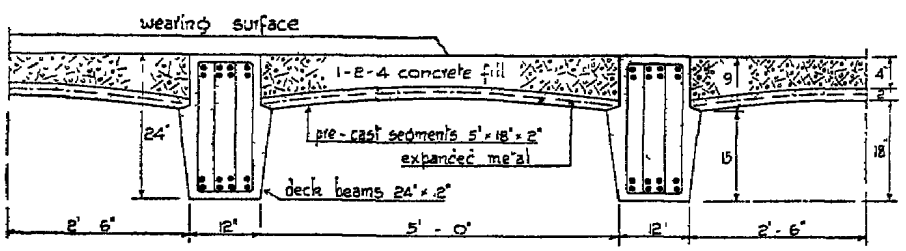
dead load moment
$M_1 = (4{\cdot}5 \times 9) + (9 \times 3)$ = 67 5 ft tons

total live load from each deck beam :
carriageway
= 10 × 6 × 280/2240 = 7 5 tons.
footwalk = 1·5 ton.
total dead plus live load per deck beam = 9 0 tons

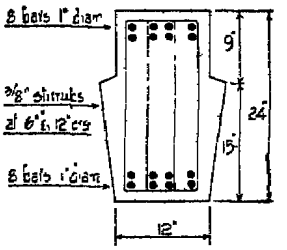
REINFORCED CONCRETE ROAD BRIDGE TO CARRY HEAVY TRANSPORT



REINFORCEMENT DETAILS FOR DECK BEAMS



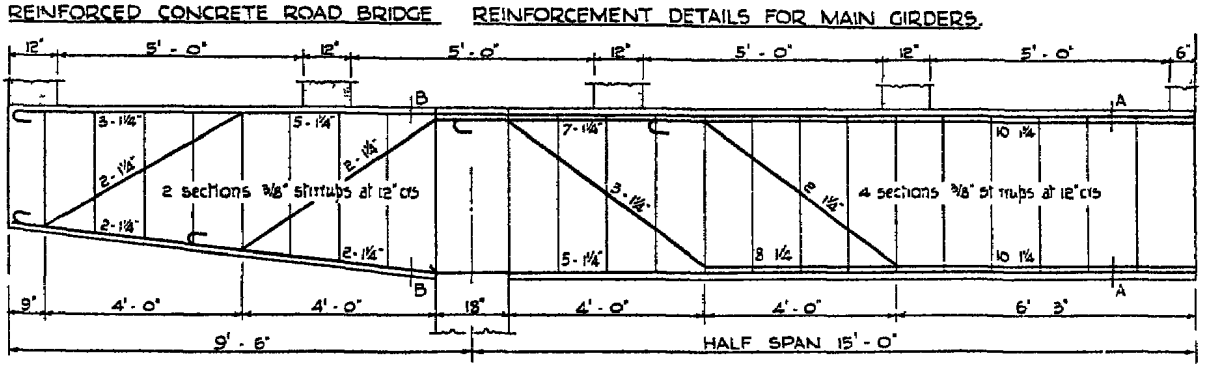
SECTION OF BRIDGE DECKING

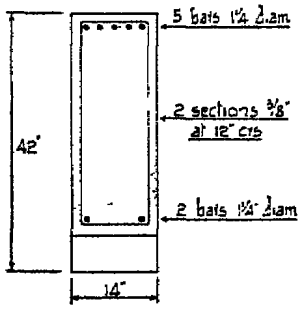


SECTION OF BEAM

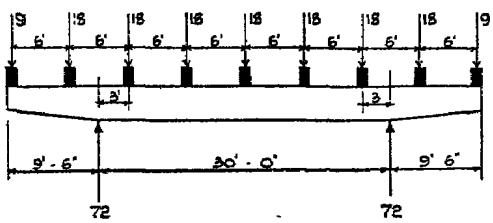
FIG 2

REINFORCED CONCRETE ROAD BRIDGE REINFORCEMENT DETAILS FOR MAIN GIRDERS.





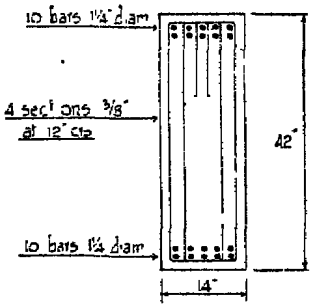
SECTION B.B



total dead load per deck beam = 9 tons

total live load per deck beam = 9 tons = 280 lbs/ft

FULL LOADING DIAGRAM FOR MAIN GIRDERS



SECTION A A

FIG 3

live load moment
$M_2 = (4{\cdot}5 \times 9) + (9 \times 3)$ = 67·5 ft. tons

total dead plus live load moment ·
$M_1 + M_2$ = 135 ft. tons.

steel area As = 135/8 × 3 = 5·6 sq. in

total shear S = 13·5 + 13·5 = 27 tons

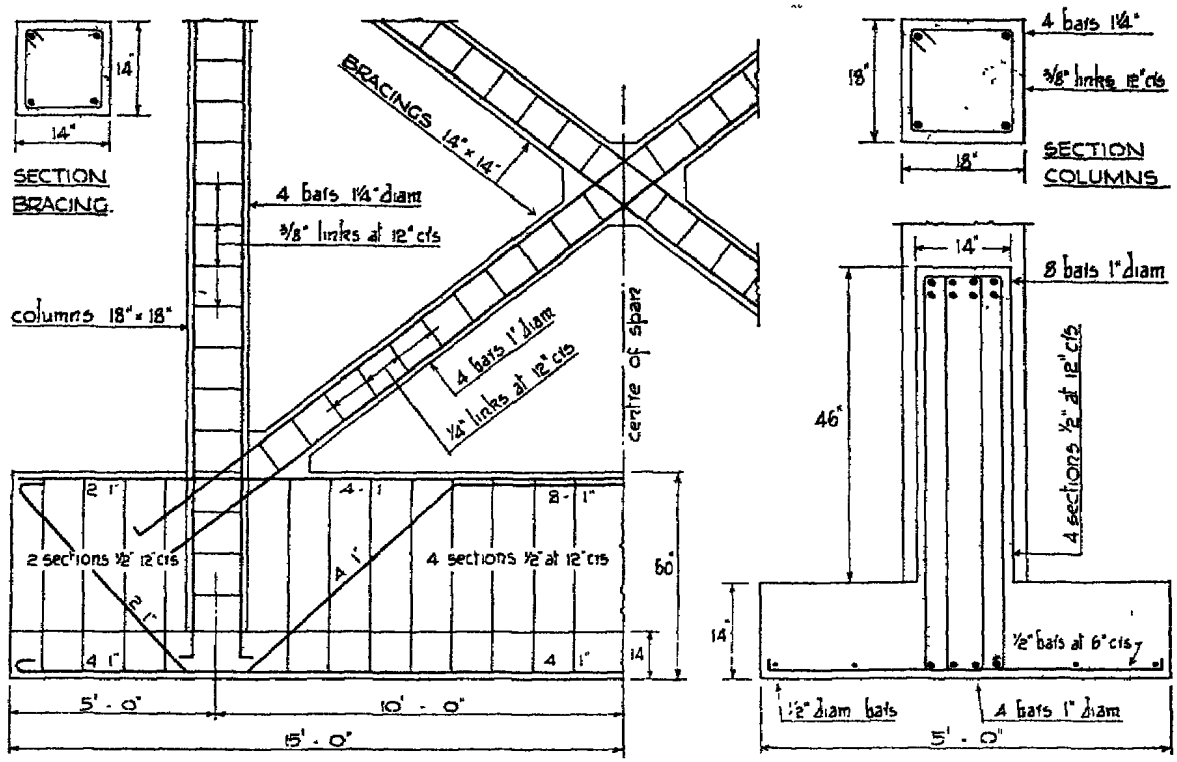
unit shear s = 27 × 2240/36 × 14 = 120 lb. per sq. in.

2 diagonals 1¼ in diam. carry :
2·45 × 8 × 2 × 36/60 = 23·5 tons.

2 sections ⅜ in diam stirrups at 12 in crs. carry .
0·22 × 8 × 36/12 = 5·3 tons.

and shear resistance provided = 28·8 tons.

REINFORCED CONCRETE ROAD BRIDGE: FOUNDATIONS, COLUMNS AND BRACINGS.



REINFORCEMENT FOR FOUNDATION BEAMS FIG 4 SECTION FOUNDATION BEAM

Centre 30-ft Portion.

Total dead and live load from each deck beam :

= 9·0 — 9·0 = 18 tons

max free moment

$M_1 = (45 \times 15) - (18 \times 12) - (18 \times 6)$ = 351 ft tons

deduct dead load cantilever moment = 67 ,, ,,

total max positive moment at mid-span = 284 ft tons

steel area As = 284/8 × 3 = 12 sq in

and 10 bars 1¼ in diam = 12 3 sq in

End panel shear S = 45 ton

and unit shear s = 45 × 2,240/36 × 14 = 200 lb per sq in

End panel shear = 45 ton

3 diagonals 1¼ in. diam carry —

3·7 × 8 × 2 × 36/60 = 35 5 ton

4 sections ⅜ in. diam stirrups at 12 in crs. carry :

0·44 × 8 × 36/12 = 10·5 ton

shear resistance provided = 46 0 ton

2nd panel shear = 45 — 18 = 27 ton

2 diagonals 1¼ in diam carry —

2·45 × 8 × 2 × 36/60 = 23·5 ton

stirrups carry 0·44 × 8 × 36/12 = 10·5 ton

shear resistance provided = 34·0 ton.

3rd panel shear = 27 — 18 = 9·0 ton

stirrups carry . 0·44 × 8 × 36/12 = 10·5 ton

General details of reinforcement for main girders are shown on Fig 3

Columns at Supports , Section 18 in × 18 in.

Total dead load from bridge decking and parapets

{8 + (5 × 2)} 8 = 144 ton

Total live load from carriageway :

49 × 20 × 280/2,240 = say 120 ton

equivalent to the load transmitted by 6 tractors each weighing 20 tons.

Total live load from foot walks :

2 × 49 × 5·5 × 100/2,240 = 24 ton

Total weight of main girders .

2 × 49 × 4·0 × 144/2,240 = 25 ton.

Weight of horizontal bracing

2 × 18·5 × 4·0 × 144/2,240 = 10 ton.

Weight of diagonal bracings :

4 × 23 × 1·4 × 144/2,240 = 8 ton

Weight of 18 in × 18 in columns

4 × 21·5 × 2 25 × 144/2,240 = 12 ton.

total load on 4 columns = 343 ton

total load per column = 86 ton.

Total wind on bridge

49 × 10 × 50/2,240 = 10 ton

overturning moment due to wind

10 × 20 = 200 ft ton

Additional load per column

200/20 × 2 = + 5 ton.

which may be neglected in view of the live load taken

Equivalent area Ae of column with 1 per cent steel = 370 sq in.

and compressive stress c = 86 × 2,240/370 = 520 lb per sq in which is satisfactory

Hence, columns 18 in × 18 in in section reinforced with 4 bars 1¼ in diam (5 sq in) and ⅜ in. diam links at 12 in centres are suitable.

Foundation Beam : 30 ft × 60 in × 14 in

Total load from 2 columns = 86 × 2 = 172 ton.

dead load foundation beam :

30 × 10 0 × 144/2,240 = 20 ton.

total load on ground = 192 ton.

area of base = 30 ft. × 5 ft = 150 sq ft

and ground pressure = 192/150 = 1 30 ton per sq. ft.

which is satisfactory

Load causing bending and shear = 172 ton.

load per linear ft = 172/30 = 5·73 ton

load per sq ft. = 5·73/5 = 1·15 ton.

Cantilever Portion :

moment M = 5·73 × 5 × 5/2 = 72 ft ton.

steel area As = 72/8 × 4·5 = 2 sq in

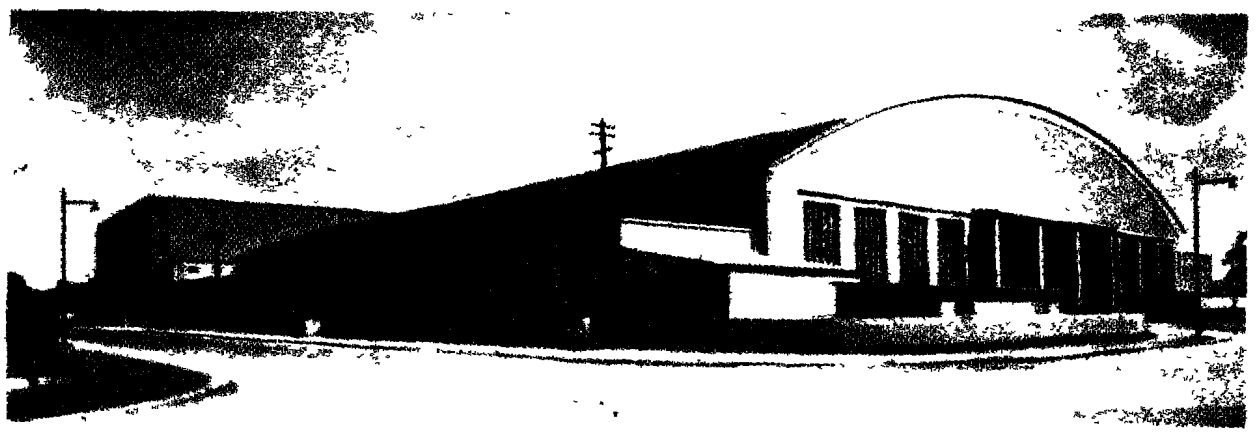
max shear S = 5·73 × 5 = 28·65 ton

unit shear s = 28·7 × 2,240/54 × 14 = 85 lb per sq. in.

(Continued on page 284)

MANCHESTER'S NEW REINFORCED CONCRETE BUS GARAGE

SHELL-ROOF CONSTRUCTION



Wythenshawe Bus Garage, Manchester.

THE Wythenshawe bus garage, a massive reinforced concrete structure which embodies features of engineering design entirely new to the Manchester area at the time of its construction during the early years of the war, is now in service for the purpose for which it was intended

Originally built for the City of Manchester Transport Department by Messrs. J A King and Co , to the design of the then city architect, Mr G Noel Hill, F.R I B A , M T P I., Messrs. Chisarc and Shell " D," Limited, being responsible for reinforced concrete design in the Chisarc system—this building—the largest in the country with this type of shell roof—was taken over by the Ministry of Aircraft Production immediately on completion in 1942, and was used by Messrs A V. Roe in the production of Lancaster aircraft

The garage, with accommodation for 100 double-decker buses was planned to serve as a depot for the corporation bus services to the Wythenshawe housing estate, and work on it was started early in 1939 It was originally anticipated that it would be completed before the end of the year, but war time conditions prevented this

Most interesting feature of its design is the " Chisarc " roof system, which has provided a roof with a clear span of 168 ft. with arched roofing slabs only 2¾ in thick, without ties, braces or internal column supports, so that it is possible to utilise every foot of floor space.

The principles of the shell-roof construction are based on the natural laws which are apparent in the shell of a bird's egg, where the " stretched skin " is also the structural element.

Saving in Materials

The two-dimensional analysis of the roof arches enables the bending moments to be reduced, due to a thrust throughout the arch In the Chisarc system the curved surface is used to transmit the loads, and internal bending moments are practically eliminated Thus the replacement of bending moments by direct forces results in a considerable reduction in the amount of material required

In the main garage section of the building a single barrel vault, of large radius, spans as a continuous beam over a series of arches set at relatively close centres The washing room and repair hall are planned as a wing set at right angles to the main building, which gives a T-shaped effect to the whole structure The washing room is roofed by means of four barrel vaults, each having a radius of 16 ft and a width of 16 ft 6 in , the four spanning a distance of 58 ft between the continuous beam and frames

For the repair hall the roof is provided by seven vaults with a radius of 22 ft 6 in and a width of 33 ft , to provide a clear span of 132 ft

The end frames for the repair hall are 232 ft in length, and allowance has been made for movement due to temperature changes by separating the centre bay from its neighbours by means of expansion joints which extend for 10 ft along the edge beams and continue down the columns to 10 ft below the soffit of the edge beams, which are themselves divided over their full length for a distance of 20 ft from the column



Interior of Main Garage, Wythenshawe.

It was not necessary to extend the expansion joints over the centre 90 ft of the span as the vaults are sufficiently flexible to take up any temperature movement The repair hall is also separated from the main garage by expansion joints

The garage roofing vaults have a radius of 180 ft and a width of 174 ft, and the roof spans as a continuous beam on arches at 42-ft centres The latter are designed as two pinned arches to carry the roof loads, as well as wind and temperature stresses The hinges are just below ground level, and the loads are carried on lead plates having a working pressure of 1,450 lb per sq in Special roof lights have been fitted throughout

Consulting engineer for Messrs Chrsarc and Shell 'D' Limited, in the roof construction was Mr H G Cousins, B Sc M Inst C E

Free floor space totalling nearly 10,000 sq yds is thus afforded in these three sections of the building, which provides a clear example of the merits of this type of reinforced concrete construction The use of concrete for a building of such dimensions will also ensure a tremendous saving in maintenance and costs

Labour-Saving Planning

The lay-out of the building is planned on the most labour-saving lines possible. On entering, vehicles will first pass through washing bays, equipped with high-level platforms, so that the buses can be cleansed from the top down without using ladders, and then the testing shops before entering the main garage

In the washing bays special sumps have been fitted to extract oil and grease from the waste water and sludge before it passes into the drains

In addition to the many usual devices installed for bus maintenance, a new feature is an installation designed to retain radiator heat in the vehicles during off-duty hours A system of flexible pipes with connections to fit to special adaptors on the radiators of the buses provides a constant flow of steam through the cooling system of each vehicle, so that water temperatures are maintained at the required peak for ease of starting In addition to its value in providing a quick gateway for the vehicle, this installation will also increase the life of the bus engines by eliminating the bad mechanical effects of a daily start with a cold engine The general heating of the garage is by the Plenum system, and special low-level extractor plant has been installed to remove the heavier-than-air fumes generated by engine exhausts

A further innovation is to be found in the inspection pits, which are lined with glazed tiles and fitted with special lighting which is designed to illuminate the engine and chassis of the vehicle without dazzling the mechanics

Provision was also made in the plans for staff dining, social and rest rooms, a milk bar and executive offices —(*With acknowledgments to "Surveyor"*)

NEW FEATURES IN GASHOLDER TANK CONSTRUCTION

USE OF SPECIAL CONCRETE CROWN SUPPORT

ENTIRELY new features have been tried out for the first time in this country in the construction of a new gasholder tank at South Gosforth for the Newcastle-on-Tyne and Gateshead Gas Co

The tank, which holds 5,000,000 gallons of water for sealing purposes, was designed by the F. C Construction Co, Ltd, to house a three-lift telescopic gasholder of 3,000,000 cub ft capacity in a manner which would permit of a minimum use of steel in the construction of the actual holder

Fixed Framing

This object has been effected by employing a fixed framing consisting of three polygonal rings of reinforced concrete beams and columns in the centre of the tank to provide a support for the crown of the holder when fully lowered

Another feature of the design was the use of permanent brick shuttering for the outer retaining walls of the tank, which has an internal diameter of 216 ft and a depth of 31 ft 9 in As sufficient timber shuttering could not be obtained for such a large-scale undertaking, it was decided to build an outer shell wall of 4½-in brick and an inner wall of 9-in brickwork

The inner wall was waterproofed on the convex face by Callenderite sheeting, which was heated on, and then given a cement rendering

Excavated material has been banked against the outer retaining wall, which rises 14 ft. above ground level, at a 2 to 1 gradient to give added strength

Walls and Floor

The walls stand on a reinforced concrete ring footing 11 ft 3 in wide and 1 ft 6 in thick, and rise from 2 ft in width at the base to 1 ft at the coping, with three intermediate steps

The floor of the tank is sloped upward at an angle of 20 deg towards a central crown and covered with reinforced slabs 15 ft by 15 ft and 6 in thick, which were cast in situ

The whole of the concrete work was carried out by Messrs Wimpey & Co

Owing to Government height restrictions the original intention to erect a four-lift holder with an overall inflated height of 170 ft had to be scrapped in favour of the present scheme, in which the holder will rise to a maximum permitted height of 110 ft above ground level

In the construction of the all-welded gasholder further steel saving was effected by a patent method of applying the steel plates diagonally, thus giving lateral strength to the structure without extra bracing being necessary

The contractors for the steelwork were the Oxley Engineering Co of Leeds

To reduce evaporation of the sealing water, oil will be poured on its surface and anti-freezing devices have been installed —(*"Municipal Journal," London*)

REINFORCED CONCRETE ROAD BRIDGE—(*Continued from page* 282)

2 diagonals 1 in diam carry :
1 6 × 8 × 2 × 4 5/6 5 = 18 ton
2 sections ½ in diam stirrups at 12 in crs carry
0 4 × 8 × 54/12 = 14 ton
shear resistance provided = 32 ton

Centre Portion
max moment M = (5 73 × 15 $^{2}/_{2}$) — (86 × 10)
= 215 ft ton
steel area As = 215/8 × 4·5 = 6 sq in
and 8 bars 1 in diam = 6 3 sq in.
total shear S = 86 — (5 × 5 73) = 57 ton
unit shear s = 57 × 2,240/54 × 14
= 170 lb per sq in
4 diags 1 in carry 3 14 × 8 × 2 × 4·5/6·5
= 35 ton.
4 sections ½ in stirrups at 12 in. crs carry.
0·785 × 8 × 54/12 = 28 ton
shear resistance provided = 63 ton

Foundation Slab. 5 ft × 14 in deep
moment M = 1 15 × 2 $^{2}/_{3}$ = 2 30 ft ton
steel area As = 2·3/8 × 0 83 = 0·34 sq in
total shear S = 1·15 × 2 × 2,240
= 5,150 lb
unit shear s = 5,150/12 × 10 = 43 lb per sq in
Transversely, place ½ in diam bars at 6 in crs.
— 0·39 sq in
longitudinally : 4 bars ½ in diam
Diagonal Bracings Section 14 in. × 14 in.
In each bracing place 4 bars 1 in diam = 3·14 sq in
and ¼ in. diam links at 12 in crs
Transverse Bracing at column heads Section 42 in. × 14 in.
Top and bottom place 4 bars 1 in. diam. = 3·14 sq in.
and 4 sections ⅜ in. diam stirrups at 12 in centres throughout

General details of reinforcement for foundation beams, columns and bracings are shown on Fig 4 —(*With acknowledgments to "The Surveyor," London.*)

STABILISING RAILROAD TRACK BY PRESSURE GROUTING

INJECTING portland cement grout under railway tracks to stabilise weak roadbed has progressed beyond the experimental stage Equipment and methods for doing a satisfactory job have been developed although improvements in methods, equipment and accessories undoubtedly will be made as experience is gained

Small jobs can be carried on as a routine maintenance with regular local track labour and equipment and extensive roadbed stabilisation can be done with extra gangs using appropriate equipment.

Successful results have been obtained with equipment and accessories improvised from material and parts found at railroad shops or those regularly carried in stock. Thus little, if any new mechanical equipment is required.

Track Conditions Suited for Pressure Grouting

Grout is most easily injected into porous ballast or where water passages are fairly well defined or where cleavage planes exist Grout will not penetrate clay or ballast so permeated with clay as to be practically embedded Between relatively clean ballast and complete embedment are all degrees of clay content The amount and distribution of clay in the pocket will determine the technique to be used and the degree of successful penetration.

Grout can be received only as air, water or other existing material is displaced If this material is liquid, as in a water pocket, it may be driven to new locations, escape through vents, or force confining barriers to yield Injecting water just prior to grout injection will wash out clay and other solid materials, and by erosion will open up passages for the entrance and uniform distribution of the grout

The Grout

Grout must cement the material between injection points into a mass which, when hardened, is practically impervious to water and strong enough to resist appreciable deformation and restore the original load-bearing capacity of the subgrade It must be uniformly smooth and cohesive enough to retain its original consistency while being injected and must contain particles of such size as will penetrate voids and passages in the water pocket. Just what proportions of cement, sand and water are to be used will depend on the physical conditions of the material to be grouted. An excess of water may result in washing sand grains clean or in separating the water and cement from the mix when under pressure. It is well to remember that while low-cost grout is desirable, an apparent saving in the cost of materials may be more than offset by increased cost of injection.

Choice of grout proportions and consistency should usually be made by trial Commence with water. If water is refused at the maximum pressure available—and it should be at least 100 p s i —it will be futile at attempt to inject grout Trials should be repeated at other depths.

After water has been accepted, further trials may be made with neat grouts commencing with 10 gal of water per bag of portland cement If this is taken freely, reduce the amount of water and add some sand—for example, 50 lb per bag of cement. If the flow indicates that a still thicker grout will be received add increments of 25 lb of sand per bag of cement until the desired consistency is secured up to a maximum of 200 lb of sand This maximum may be exceeded somewhat where the sand is extremely fine (passing a No 50 sieve and containing silt and clay).

Do not attempt to use a sand unless 100 *per cent of it will pass a No* 14 *sieve* and preferably a No 30 sieve Even finer sand is better. Usually an engine or beach sand is suitable for the common type of water pockets It is quite important that sand be screened before delivery, because it is time consuming and expensive to do this at the site The dry materials should be passed through a coarse sieve ($\frac{1}{4}$-in mesh) just before the mixing operation to remove lumps of sand or cement and foreign materials

Quantities of materials required for pressure grouting cannot be estimated accurately The quantity of grout accepted will vary from that requiring 1/10 bag of cement per track foot to 1 bag per track foot The quantities listed below are, therefore, rough approximations presented only as a reasonable guide



A short, bad squeeze or a type oftentimes encountered.

For 1,000 track feet
Portland cement at $\frac{1}{2}$ sack per track foot equals 500 bags
Fine sand at 100 lb per track foot equals 50 tons
Water (including waste) at 15 gal per bag of cement equals 7,500 gal.

Estimating a rate of progress for one small gang at 100 track feet per day, at least 10 per cent of the above quantities should be kept ahead at the location being treated

Labour

The gang required on a track grouting job may comprise the regular track forces supplemented at least on initial jobs by one or two bridge and building men accustomed to handling mixers and other mechanical equipment The track foreman with a gang of four to six men should be able to handle the work after some experience has been gained If much work is on the programme it is advisable to train a special gang for the purpose and provide a work car and mixing and grouting equipment mounted on an off-track, self-propelled grout mixer and pressure unit that can be set in any position along the shoulder

On the small job, four to six men are first assigned to installing injection points After this is done, the same men set up equipment and distribute material, including water, where it is convenient for use When grouting is under way, two men serve the mixer and pressure tank, one operates the valves and gates, two are stationed at the injection point, and another man performs roving duties such as helping pull injection points, check track level and assist in relieving line plugs

Where rate of progress warrants, additional men may be assigned to install grout points ahead and pull them after grout application is finished.

Equipment

Mixing and grout injecting equipment should be of liberal size so that it will not restrict grouting operations Essential spare parts should be kept on hand to prevent shutdowns

The amount of grout accepted by the roadbed will vary widely It is important that enough grout be on hand to keep the gang going at a reasonable rate As a rough guide, it is suggested that 3 cu ft of grout per track foot be considered as a maximum, and mixer and injector should be able to furnish and inject 90 cu ft of grout per hour, or 1 5 cu ft per minute. The mixer tank or grout box should hold about a 10-minute supply of grout or 15 cu ft If the mixer is a continuous type a much smaller storage capacity will suffice

To insure thorough and uniform incorporation of all materials the mechanical action should be sufficiently violent to pulverise all lumps that may form and to agitate equally the material in all portions



Driving spuds and points with sledges.



Driving spuds and points with air hammers.



Driving point with pipe sleeve.

of the mixing chamber Preferably a continuous supply of grout should be available at all times during the grouting operation to enable the track gang to work without interruption. This may be done by providing twin mixing compartments, one of which should be mixing while the other is discharging, or duplicate mixers may be used

Mixing may be done in numerous ways with hand tools in a mortar box; with paddles in a tub, or by hand with a crank-operated vertical shaft with blades in a 50 gal drum Another method, used successfully for many years in the Ransome pressure grouter, mixes the grout by letting jets of air bubble upward from the bottom of the tank Premixing in a mortar box may be done with mortar hoes and straight-backed rakes with the teeth up The grout should be strained through a ¼-in mesh screen in order to get rid of lumps and foreign material before passing through the injector

Effective grouting can be done with pneumatic or hydraulic pressure Pneumatic equipment has been used most generally; however, hydraulic equipment has certain advantages that are bringing it into increasing favour

Pneumatic equipment should be simple and portable The air supply may be a compressor of the type and capacity furnished for tamping ballast with air tools, or if the work is in a terminal the air may be obtained from the supply for operating switches The pressure tank should have an airtight charging door and appropriate connections for controlling the air supply and discharging the grout Pressure tanks may often be improvised from tanks on hand at the railway shops or adapted from units used for similar purposes, such as the chamber of a Cement Gun unit.

Pneumatic equipment should be limited to pressures not much in excess of 100 p.s.i. Air pressure sustained a considerable period after relief is provided may prove annoying, particularly if too high a pressure is maintained in the tank. Track may be raised objectionably and a considerable amount of air may be injected after the grout has cleared the line

It is important that all grouting equipment and particularly the pressure tank and connections be kept clean and free from accumulation of hardened grout. All parts of the equipment should be readily accessible for cleaning and the tank should be constructed to encourage uniform agitation and have no dead zones where grout can deposit and harden

Hydraulic equipment also should be simple and portable. In cases where voids and passages in the material to be grouted are small or where water must first be displaced from fine-textured materials, pressure from 150 to 300 p s i. may be necessary Hydraulic pressure is ideal for this purpose as pressure builds up gradually until grout is accepted and when flow commences no disruptive violence results After the initial resistance has been overcome and water or grout starts flowing, the pressure needed for continued flow usually drops off considerably Distribution of grout proceeds steadily as governed by local resistance When grout breaks through weak places or finds a vent, hydraulic pressure is immediately reduced.

A single-acting plunger pump operated by an air or gas-driven motor, or a hand-operated duplex plunger pump can be used effectively. A mudjack, employed in bringing concrete pavement slabs to grade is also well adapted for grouting track.

Auxiliary Equipment

The usual track, carpenters', and plumbers' tools, pipe lines and connections, water barrels, blocking and other miscellaneous items available to track gangs are necessary. A small hand or gas-driven pump may be needed if water is obtained from a stream or ditch.

The grout discharge line should be as straight and have as few fittings and valves as possible All parts should be designed to discourage line plugs Connections should be of a type quickly broken and blow-offs or by-passes should be provided to quickly clear the line and permit easy cleaning A 1¼-in pipe line has been found satisfactory For a short distance from the pressure tank this may be an iron pipe and to this is attached a flexible rubber hose such as a steam hose or the type used for Gunite. The hose should not be too long because the tendency to plug and the difficulty of cleaning increase with the length.

Discharge lines have ranged from 30 to 120 ft. long The shorter length permits serving about 50 ft. feet of track before moving and is adequate for good co-ordination between the men at the injection points and the grout plant. Since the longer length is more subject to plugging, extremely fine sand and higher pressure will probably be necessary but it makes possible the treatment of over 200 ft of track from each set-up

For controlling the flow of grout a valve may be provided on the discharge line at the injection point A satisfactory valve for this purpose is an angle valve commonly used on railroad train air brakes Every effort, however, should be made to eliminate valves and fittings not indispensable. Usually all necessary control may be accomplished by providing a quick-acting coupling at the injection point The connection of the discharge line to the grout point should be simple and permit rapid coupling and uncoupling A standard air-hose coupling, railroad air-brake hose coupling, or a standard threaded pipe union is satisfactory for the purpose.

Provision should be made to quickly relieve stoppages in the discharge line. This is important. Uncoupling at the injection point or opening a blow-off



Tools used to instal injection points. The square spud bar is first driven either by sledge or jackhammer and pulled. Into the hole thus formed the injection point is set and driven with a weighted rod. The short cylinder on the rod below the driving weight is screwed on the injection point and serves as a driving cap.

valve at the point are usually effective. The grout is permitted to blow until flow is established. Grout is wasted in a ditch or caught in a bucket. In stubborn cases air is forced back through the line from the point to the grout chamber. A separate air line connected to a valve located just back of the injection point has been satisfactorily used for this purpose

Injection points must withstand severe treatment in driving and pulling Large pipes offer too much resistance to driving, small pipes and pipes with thin walls are too flexible. Standard double strength 1¼-in. diameter iron pipe has been commonly used and is usually available at railway shops. If 1-in. pipe is used an appropriate bushing will be needed to connect to the 1¼-in. discharge line. For the average job 25 to 50 injection points about 5 ft long will be needed. Several threaded nipples with couplings, ranging in length from 18-in to about 4 ft should be available to accommodate the few places where greater depths must be reached. The injection points are left open at the tip and threaded at the top to receive a standard coupling or a threaded driving cap.

Where ballast permits easy driving the tip of the pipe may have a loose-fitting metal pilot plug (usually an old bolt head or rivet head). A cleaning rod, 1-in. diameter and 6 to 8 ft. long, with a loop handle at one end is used to drive the plug out and free the tip.

In long-seasoned roadbed the ballast usually offers high resistance to driving the injection points It is therefore recommended that a solid steel spud bar of slightly larger size than the injection point be first driven to the desired depth and then withdrawn, leaving a hole to receive the injection point. As the spud bar is driven it should be kept loose by twisting with large Stillson wrenches or other effective means. The injection points are then set in the holes thus formed and easily driven to any depth desired. No pilot plugs in these injection points are needed, as any material that plugs them may be pushed out with the cleaning rod.

Both spuds and injection points may be driven either by hand or power The heads of spuds withstand the abuse of driving without much damage. Driving heads are needed for driving injection points. For light driving, pipe couplings are often adequate and can be reused a number of times. Where heavy resistance is encountered special heavy threaded metal caps may be needed. Driving by hand may be accomplished with 12 lb. to 18 lb. sledges, weighted pipe sleeves (40 to 50 lb), or weighted rods worked inside the pipe through a drilled hole in the driving head. When air is available a jackhammer or similar percussion hammer may be employed. The driving head of the grout point should be shaped to receive the cup of the hammer. The upper end of a broken



Pulling a point with a chain and claw bar or track jack.

drill bit with the shoulder bearing against a standard pipe coupling on the upper end of the point has been effectively used for this purpose.

The pulling of spuds and injection points is another operation in which suitable methods and tools will expedite the job. Pulling may be done with large Stillson pipe wrenches or a chain and claw bar In more stubborn track, jacks reacting against thick iron plates with holes to slip over the points may be effective. A steel pinch plate about ¾-in. thick having a hole slightly larger than the point is a useful device. With the point passing through it, the pinch plate is placed on the ground With a tie plate for a fulcrum and a claw bar for a lever an effective uplift is obtained, and by pumping the claw bar up and down, a rapid rachet action may be produced.

Summary of Equipment and Accessories

Air compressor (portable preferred), capacity 75 to 100 cu. ft per minute at pressures up to 100 p s i.

Grout mixer, type determined by local conditions as discussed

30 *to* 100 *ft. of rubber discharge hose* such as 1¼-in steam hose with fittings to connect to pressure tank and injection points.

25 *to* 50 *injection points*, 1¼-in. double strength iron pipe, each about 5 ft long with standard pipe threads at one end.

6 *to* 12 *pipe nipples* of various lengths from 12 to 48-in, threaded both ends and equipped with couplings.

50 *to* 100 *metal plugs* such as scrap rivet or bolt heads to fit tip of injection point loosely If spud bars are used these may be omitted

¼-*in screen* to remove foreign material and lumps from sand, cement and the grout

2 *solid steel spud bars*, 1⅜-in diameter or 1⅜-in. square, and 5 to 6 ft. long.

2 *steel pinch plates*, 5 × 5 × ¾-*in*, with holes to loosely fit spud and injection point

Small water pump (either gas or hand operated) if water is to be obtained from streams or ditches.

Water barrels as determined by job conditions

2 *hand sledges*, one 12 lb and one 18 lb

Air tools, to include one or more jackhammers or equivalent with air lines to operate them

2 *driving caps* to fit injection points

1⅜-*in rock drill bit* if unusually hard ballast is anticipated

Control valves on grout discharge line and blow-off, if desired

Carpenter, plumbing and track tools and equipment usually assigned to track gangs using air equipment

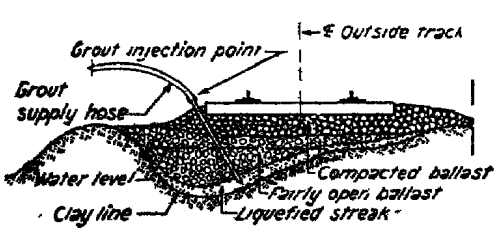
Timber blocking

Spare fittings, gaskets and minor critical parts subject to quick wear or breakage.

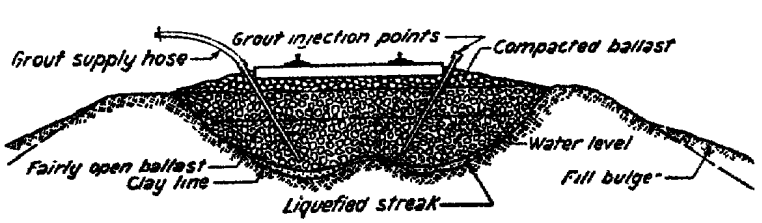
Fuel and Oil.

Procedure.

Occasional test pits or cross trenches to determine the nature of the material under the track at the more troublesome spots are desirable A practical advantage of the pressure grouting method is that varying under-track conditions are accurately sensed by the resistance



Double Track. Treatment usually required only under outside rail, because lateral support is weakest along the shoulders.



Single Track. On fills, bulges usually appear further from the track than in cuts, and often down the slope. Treatment from both sides usually needed for single track.

encountered as the points are being driven Usually the top portion of the ballast is compact, making the early resistance hard, then the point passes through a zone of less resistance until the clay line in the bottom of the water pocket is reached, and as the point enters the clay, resistance again changes Since grout cannot be forced into clay, driving should stop at this clay line If driven deeper the point should be pulled back free of the clay before grouting is commenced.

In case the injection point is too short to reach the clay line, it may be lengthened by adding a nipple, and driven deeper. This lengthening is seldom necessary because the pocket is usually so enclosed that the injected grout displaces the liquid material at depths well below the tip of the injection point.

Where doubt exists as to the ability of the water pocket to take grout, it is advisable to install a few widely spaced injection points and test them by injecting water under pressure. If water is not received at the first attempt the point should be pulled up 6-in and another attempt made These attempts should be repeated at successively lesser depths up to 2 ft under the base of rail If water is then refused at the maximum pressure available should be not less than 100 psi) grout would not be accepted at this point If the water test shows favourable conditions, enough injection points should then be installed to keep about one-half day's work ahead of the grouting operation

The injection points should be spaced opposite every second crib and driven at the outer ends of ties at an angle to bring the tip approximately under the rail The angle of driving is not a critical one and may vary from 45 to 15 deg with the vertical but adequate clearance should be given between the points and passing trains In case injection points are installed between rails, they may be set either vertically or at an angle, but the top of the pipes must be below the top of rail.

Before commencing grouting, a track level board should be placed across the rails opposite the point being grouted to guard against the track heaving out of cross level Promptness in cutting the air supply immediately after grout clears the point will forestall much heaving of track

An orderly sequence of grout injections is advisable for efficient operation and the recording of construction data. Work should start at one end of the pocket and move progressively ahead. Injecting water into 10 to 12 points immediately preceding grouting has a beneficial washing and lubricating effect. Where this water flows or gushes from points driven some distance away it may be well to permit the flow to continue for the purpose of creating a channel. When and how long to do this is a matter of judgment

To control the upper limit of the injected grout, pits may be dug in the cribs or just beyond the tie ends, with their bottoms at the level above which grout should not rise Usually this is not necessary because the top ballast contains too much fine material to permit much grout to penetrate it except in occasional chimneys or porous areas The affected ballast is usually only weakly cemented and may be easily loosened with tamping tools If break-throughs occur in the top ballast or in the shoulders or between tracks, grouting should be transferred to another point Later, after this grout has stiffened another injection should be tried This may often be done in less than an hour.



Injecting water is usually advisable to test the possibility of receiving grout, and also when the ballast is very dirty it facilitates the distribution of grout.



For injecting grout between rails the point is driven vertically with its top below top of rail.

Grout injection should be continued at a point as long as grout is being accepted, provided there are no break-throughs which waste grout or the track is not being heaved objectionably. If grout is refused at a lower depth the point should be progressively raised until grout is taken Grout welling up or spouting from other points indicates that a channel has been opened between their tips. The venting pipe should then be plugged, either by screwing on a standard pipe cap or by driving in a wooden plug, and grout injection continued at the first point. Grout appearing in the side of a ditch or down the slope of an embankment indicates that the grout has found a path of least resistance. Sometimes this flow may be checked by shovelling and tamping dirt at the vent. If this is not effective the injection should be changed to a new point Should grout blow back up the outside of the pipe the flow may be checked by stamping fine material around the pipe, also by letting it rest a few minutes while injecting grout at another point

If grout flow ceases while pressure is on, prompt location of the plugs and clearing the line will obviate much



Grout has cut channel between bottoms of pipes. Pipes should be capped or plugged when this happens and grouting continued at the first point. Note track level board.

trouble. Pull the point progressively to higher elevation while the grout is under pressure If grout will not flow, then open blow-off valve or uncouple at the injection point to clear the line If the line clears, transfer the connection to the next injection point while pulling and clearing the plugged point. If the plug is in the line, pounding with a wooden mallet may clear it, or if this fails apply back pressure This is a good time to check the consistency of the grout Plugs composed mostly of sand indicate that the amount of sand should be reduced and possibly that a much finer sand should be substituted Do not add water to the grout under these conditions

Should there be any doubt as to the effectiveness of the grouting at any location, another injection may be tried after rodding out the point or additional injection may be made through new points installed the next day After the second injection the point should be rodded out below its tip and grouting attempted at a lower elevation before the point is pulled up. Other locations for application of grout should be tried as appear desirable.

Injection points should be rodded out before grout hardens, and should not be permitted to remain in place more than a few hours after grouting because the setting of the grout and the compaction due to traffic may make pulling difficult. In case passing trains cause grout to spout from the points, these should be capped or plugged not only to prevent loss of grout but also to take advantage of the wider distribution of grout caused by pressure from the trains.

Before considering the treatment at any location completed, the record should be reviewed to be sure that all points have been grouted before moving to a new location

Following grouting, the track is brought to final line and surface should require little or no further attention other than normal maintenance on good track. It is advisable to mark the treated locations with stakes so that comparative maintenance records may be kept. Equipment and accessories should then be carefully cleaned and checked so that worn and broken parts may be renewed and spares restocked for use at a new location —(*With acknowledgments to "Constructional Review."*)

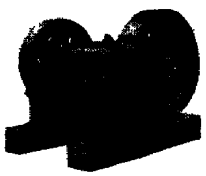
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Secondary breakers single toggle. Sizes from 12×8 to 42×15.



Granulators. Sizes from 12×6 to 42×6.

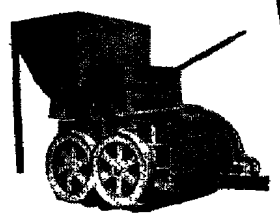
Now is the time for quarry concerns to secure a lead by buying superior equipment for producing aggregates for road-making, concrete constructions, tarred and bituminous stone and chippings.

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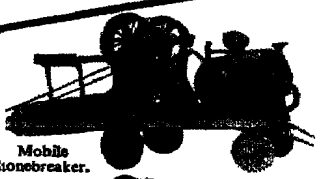


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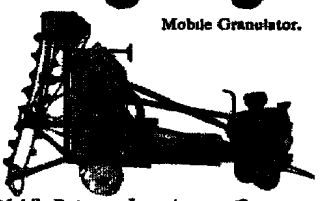
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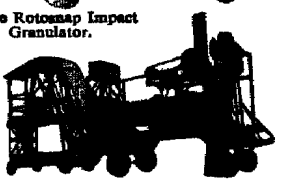
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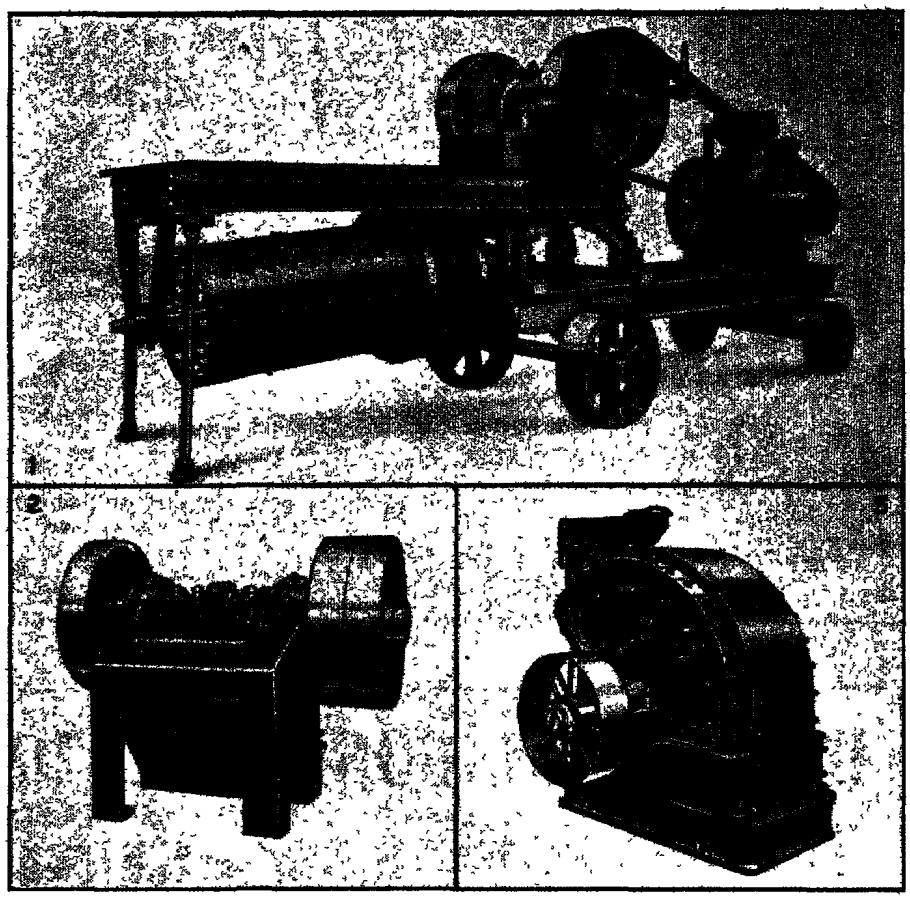
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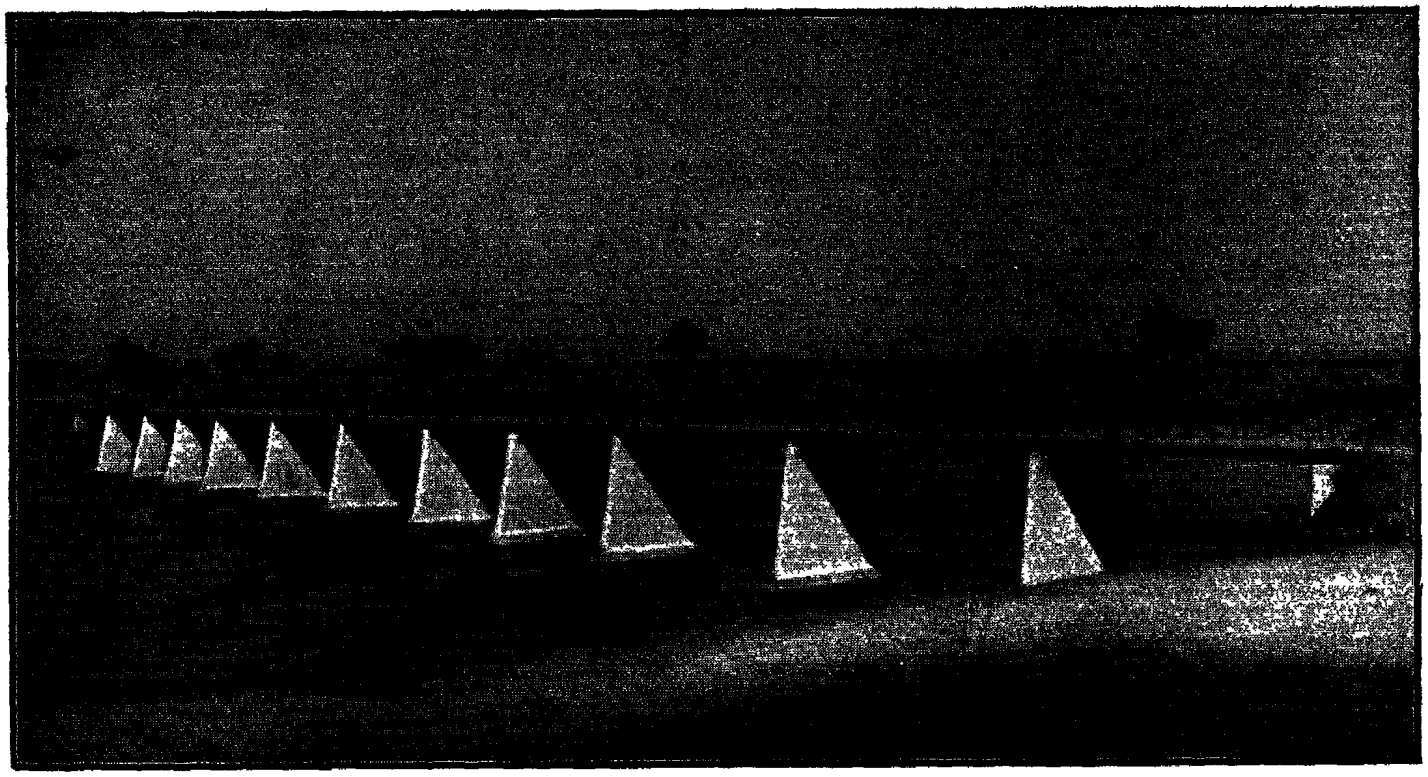
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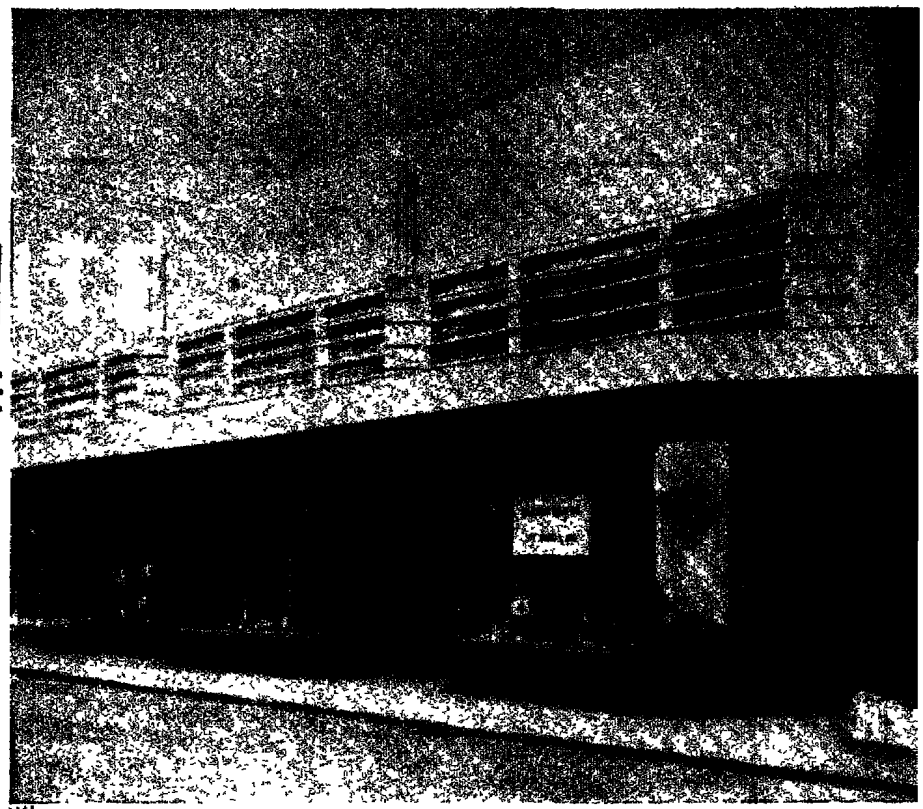
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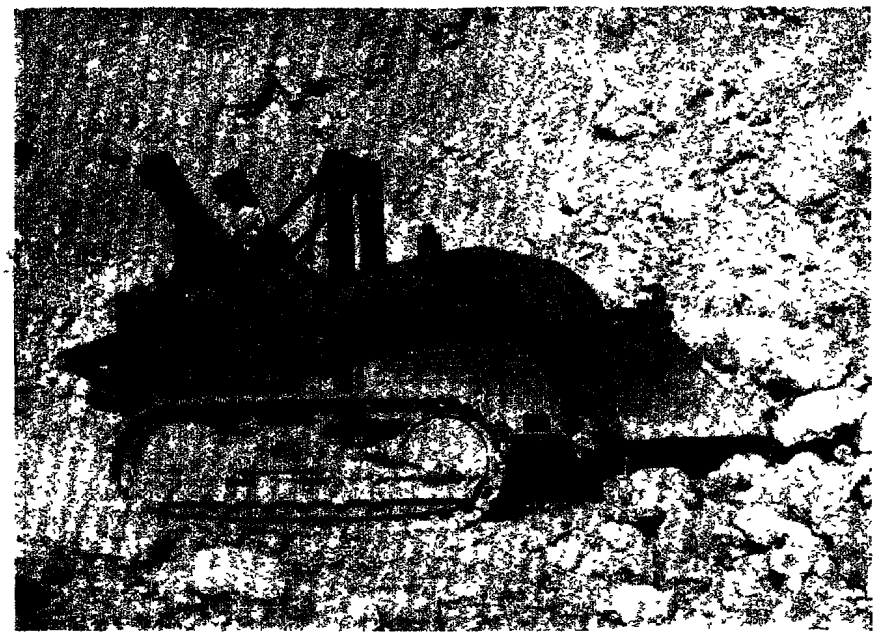
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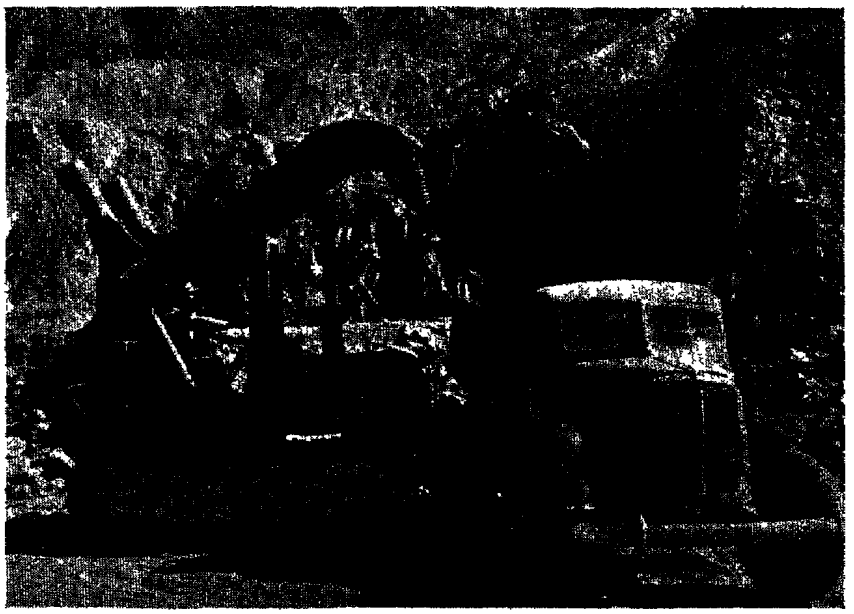
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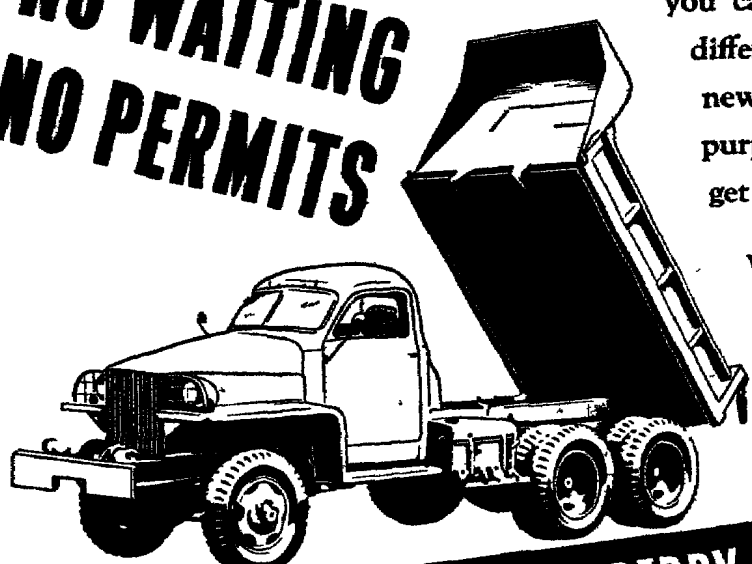
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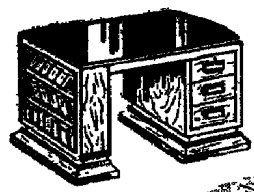
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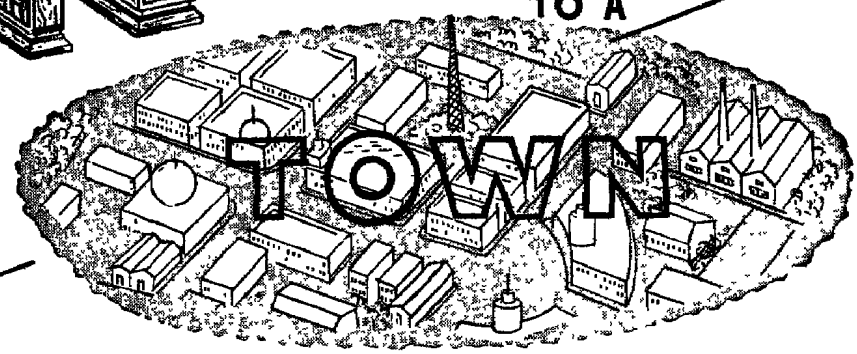
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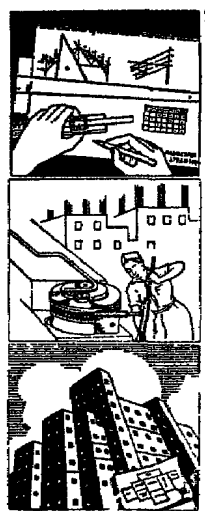
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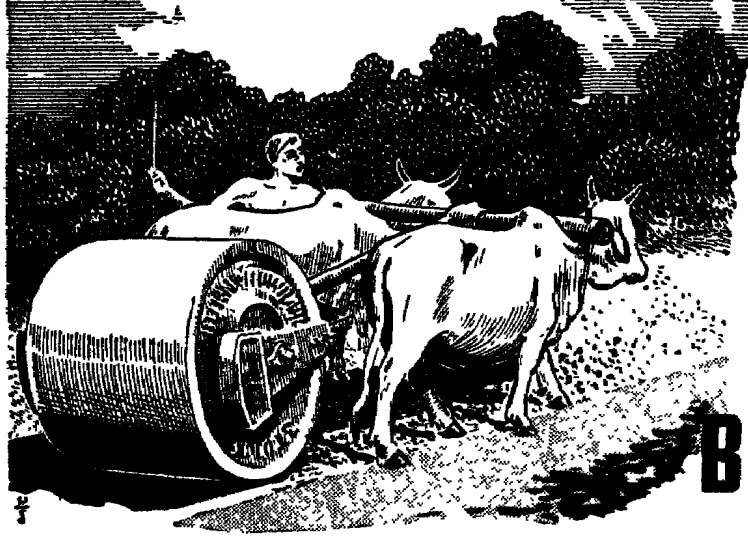
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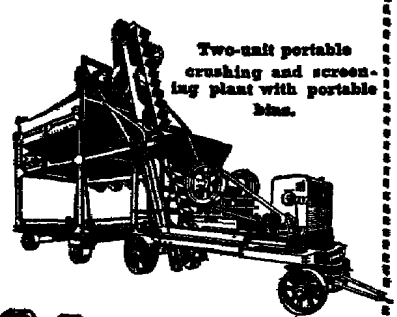
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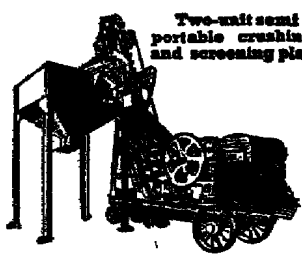
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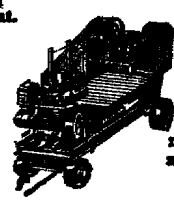
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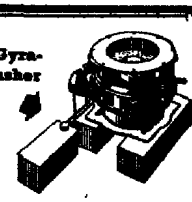
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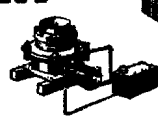
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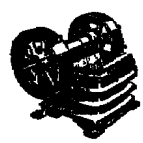
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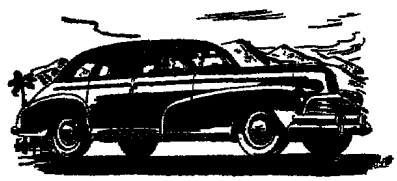
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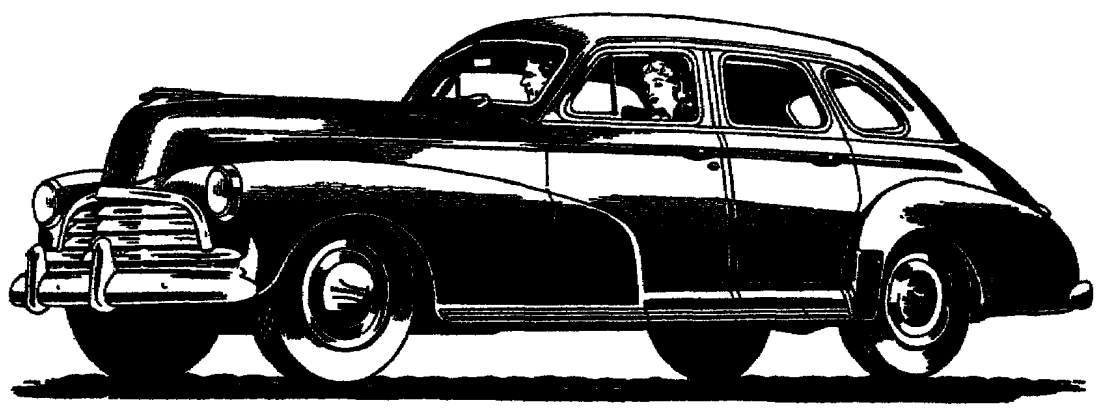
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LAHORE AREA :—

ABBOTTABAD
R. S Havelian
ABDUL HAKIM
ABOHAR
ADAMPORE DOABA
AHMADGARH
AKALGARH
ALAL
ALACHOR
AMARSIDHU
(R S Kot Lalkpat)
AMRITSAR
ARIFWALA
ATARI
BADAMI BAGH
BAIJNATH PAPROLA
BADDOMALHI
BAHRAM
BAHAWALNAGAR
BAHAWALPUR
BANGA
BANNU
BARIWALA
BATALA
BEGOWAL GHARTAL
BEAS
BHAGTANWALA
BHAKKAR
BHALWAL
BHATINDA
BHAUN
BHERA
BHOGPUR SIRWAL
BASSI PATHANAN
BASAL
BASIRPUR
CAMBELLPUR
CHABIANA
CHACHRAN
CHAK JHUMRA
CHAK AMRU
CHAK KANRO
CHAKWAL
CHAWINDA
CHANANWALA
CHANGAMANGA
CHENAB WEST BANK
CHHEHARTA
CHICHAWATNI ROAD
CHINIOT
CHIRTHEN
CHUHARKANA
CHUTIANA
DALHOUSIE
R. S. Pathankot
DAULATALA
DASKA
DASUYA
DERA BABA NANAK
DERA ISMAIL KHAN
R. S. Duryakhan
DERA GHAZI KHAN
DERA NAWAB SAHIB
DARBARSAHIB
DHARAMPUR
DHARIWAL
DHILWAN
DHUDIAL
DHURI
DHABAN SINGH
DINA
DURGAI
DINANAGAR
DINGA
DOBAHA
DUNGA BUNGA
DUNYAPUR
EMINABAD
FARIDKOT
FATEHGARH CHURIAN
FAZILKA
FATEJANG
FEROZEPUR CITY & CANTT
GARESHANKAR
GAKHAR
GAKHOA
GHAGGAR
GIDDARBAHA
GONIANA
GORAYA
GUJARKHAN
GOBINDGARH
GOJRA
GUJRANWALA
GUJRAT
GUL IMAM
GURDASPUR
GURU HARSHAHAI
GUNJAYAB
HAFIZABAD
HARUNABAD
HARIPUR HAZARA
HASILPUR
HINDUMALKOT
HIRA SING
HOSHIARPUR
HARAPPA ROAD
HASAN ABDAL
ISAKHEL
INJRA
JAGRAON
JAHANGIR ROAD
JAHANIA
JALON DOABA
JAMMU
JALALABAD
JALLO
JAITU
JANDIALA
JANIWALA
JARANWALA
JHANG MAGHIANA
JHELUM
JOGINDERNAGAR
JULLUNDER CITY & CANTT
JUTOGH
JWALAMUKHI ROAD
KHEWRA
KABUL
R. S Chaman & Peshawar
KABUL RIVER
KAMALIA
KALAKE
KALKA
KALABAGH
KALLUR KOT
KAMOKE
KANGANPUR
KANGRA
KAPURTHALA
KARTARPUR
KAROR
KASAULI
R S Kalka
KHAIRABAD
KHUSHAB
KASUR
KHANEWAL
KHAROR PAKKA
KOT RADHAKISHEN
KANDAGHAT
KUNDIAN
KHANNA
KHANPUR
KHASA
KOT ADU
KHARIAN
KILA RAIPUR
KOHAT
KOT KAPURA
KOT LAKHPAT
KURALI
LAHORE & LAHORE CANTT.
LAKHEWALI
LAKIMARWAT
LALAMUSA
LALIAN
LODHRAN
LOHIAN KHAS
LEIAH
LUDHIANA
LYALLPUR
MACLEOD GUNJ ROAD
MAGHARWAL
MALISI
MAJITHA
MALAKWAL
MALERKOTLA
MAKHU
MALOUT
MALSIAN SHAKOT
MANDI BAHAUDDIN
MANDRA
MANSA
MARH BALOCHAN
MANDI BUREWALA
MANDI STATE
MANGWAL
MARDAN
MAMU KANJAN
MAUR
MAINWALI
MIRPUR,
R S. Sirai Alamgir
MIRAN SAHIB
MOGA
MOGHALPURA
MONTGOMERY
MORINDA
MUKERIAN
MUKTSAR (B B &C.I.)
MULLANPUR
MULTAN CITY & CANTT.
MURREE
R S. Rawalpindi.
MUZZAFFARGARH
MURIDKE
MIANCHANNU
MINCHINABAD
MIRAN SAHIB
NAKODAR
NANKANA SAHIB
NARANG
NAROWAL
NAWANSHAHR DOABA
NOWSHERA
NURKOT
NURMAHAL
NURPORE ROAD
NAGROTA
OKARA
PALAMPUR
PARACHINAR *(Via Thal)*
PASRUR
PATHANKOT
PAKPATTAN
PATTI
PINDIGHEB
PINJOUR
PATTOKI
PESHAWAR
PHAGWARA
PHILLAUR
PHULERWAN
PIRMAHAL
PIND DADAN KHA
PIPLAN
QAIDAN
QILLA SHEIKHUPURA
QILA SOBHA SINGH
RAHON
RANBIRSINGHPURA
RAIKOT *(R. S. Mullanpur)*
RAHWALI
RAIWIND
RAHIMYARKHAN
RAJA JANG
RANBIR SINGHPURA
RORANWALA
RAWALPINDI
RENALA KHURD
RUPAR
RURALA ROAD
SADIKABAD
SAMBRIAL
SAILA KHURD
SANAHWAL
SANGLA HILL
SILLANWALI
SARA ALAMGIR
SARGODHA
SARNA
SATNAUR BADESRON
SHAH ALAM
SHAHDARA
SHARAKPUR
(R S Shahdara)
SHAHPUR SADAR
SHAH CHAURASI
SHORKOT ROAD
SHUJABAD
SIALKOT
SIHALA
SIMLA
SIRHIND
SOLON
SUKHO
SULTANPUR LODRI
SUKHEKE
SRINAGAR
R. S. Havelian or Rawalpindi.
TALAGANG *(R S. Chakwal)*
TALWANDI
TARN TARAN
TANDA URMAR
TANDIANWALA
TANK
TAXILA
THERI SANSI
TEHSIL SHAKARGARH
THABALKE
THAL
TOBA TEK SINGH
VEHARI
VERKA
WAZIRABAD
WARBURTON
WASAWEWALA

NEW DELHI AREA :—

ACHNERA
AGRA
ALAL
ALIGARH
AMROHA
AMBALA CITY
AMBALA CANTT
ATRAULI ROAD
BABRALA
BARHAN
BARARA
BARNALA
BAGHPAT ROAD
BAH *(R S Belangan.j)*
BAHADURGARH
BAHJOI
BARETA
BASIKIRATPUR
BEGAMABAD
BHIWANI
BHUCHCHU
BALLABGARH
BARAUT
BUDHLADA
BULANDSHAHR
BIJNOR
BRINDABAN
CHANDAUSI
CHANDPUR SIAU
CHHINTANWALA
CHOLA
DADRI
DALMIA-DADRI
DASNA
DIBAI
DEHRA DUN
DELHI
DELHI SHAHDRA
DEOBAND
DANKAUR
DAURALA
DHAMPUR
DHANARI
DOIWALA
ETMADPUR
FARIDABAD
FEROZABAD
GANAUR
GANGOH
GARHMUKTESAR
GHAZIABAD
GHARAUNDA
GOT
GOHANA
GURGAON
GULAOTHI
HALDAUR
HANSI
HARDWAR
HAPUR
HATHRAS
HISSAR
HASANPUR *(R. S. Gajraula)*
JAGADHRI
JAKHAL
JALESAR ROAD
JAWALAPUR
JIND CITY
JULANA
KALANWALI
KANTH
KANDHLA
KAURARA
KHATAULI
KHEKRA
KHURJA
KARNAL
KAITHAL
KASHIPUR
KASIMPUR KHERI
KOSI KALAN
KOSLI
KOTDWARA
KURUKSHETRA
LEHRAGAGA
LHAKSAR
LOHARU
MANDI DHANAURA
MADLAUDA
MANGLORE
MANSURPUR
MEERUT
MILAK
MORADABAD
MOWANA MANDI O/A
MUNDLANA
MURADNAGAR
MUSSAN
MUSTAFABAD
MUTTRA
MUZAFFARNAGAR
NAJIBABAD
NAGARIA SADAT
NAGINA
NABHA
NANAUTA
NARELA
NARWANA
PALWAL
PANIPAT
PATIALA
PILKHUWA
RAIWALA
RAJA-KA-SAHASPUR
RAJGHAT-NARORA
RAJPURA
RAMAN
RAMNAGAR
RAMPUR
RAMPUR MANHYARAN
RAMPURA PHUL
RAYA
REWARI
ROHANA KALAN
ROHTAK
ROORKEE
RIKHIKESH
SAFIDON
SAHARANPUR
SAMPLA
SANGRUR
SASNI
SUNAM
SAMALKHA
SAMBHAL HATIM SARAI
SARSAWA
SEOHARA
SHIKOHABAD
SHAHABAD MARKANDA
SHAMLI
SIRSA
SONEPAT
SIMBHAOLI
TAPA
TAPRI
TARAWARI
THANA BHAWAN
THANESAR
TUNDLA
TOHANA
UCHANA
UKLANA

CAWNPORE AREA :—

ACHALGANJ
ACHHALDA
AGSAULI
AHIMANPUR
AHRAURA ROAD
AJGAIN
AJODHYA
AKBARPUR
ALIGANJ
ALLAHABAD
AMETHI
AMOUSI
ANJHI
ANUPGANJ
AONLA
ASAFPUR
ATARRA
AURAIYA
AUNRIHAR
AZAMGARH
BABHNAN
BACHRAWAN
BADLAPUR
BADSHAHPUR
BAHERI
BAHRAICH
BAHRAMGHAT
BAISWARA
BAITALPUR
BAKSHA
BALAMAU
BALLIA
BALRAMPUR
BAMIANA
BANBASSA
BANDA
BANGAR MAU
BARA BANKI
BAREILLY
BARHAJ BAZAR
BARHNI
BARHALGANJ
BASAI
BASHARATGANJ
BASTI
BELLARRIN
BELTHARA ROAD
BENARES CANTT.
BENARES CITY
BEWAR
BHADOHI
BHARTHANA
BHATPARRAN
BHATNI
BHITAURA
BHITI
BHONGAON
BIJAURA
BILGRAM
BILPUR
BAINDKI ROAD
BISALPUR
BISWAN
BODARWAR
BRIDGMANGANJ
BUDAUN
BURHWAL
CAMPIERGANJ
CAPTAINGANJ
CAWNPORE
CHANDAULI MANJHWAR
CHANDWAK
CHAURICHURA
CHEHTAUNI
CHHIBRAMAU
CHILBILA
CHILK
CHILWARIA
CHIRGAON
CHIT-BARAGAON
CHOWKGHAT
CHUNAR
COLONELGANJ
DARYABAD
DARYAOGANJ
DATIA
DHEENA
DIBIAPUR
DILDAR NAGAR
DOBHI
DOHRIGHAT
DULAHPUR
ETAH
ETAWAH
FARRUKHABAD
FATEHGARH
FATEHPUR
FYZABAD
GAHMAR
GANJDUNDWARA
GAURIGANJ
GHAZIPUR
GHOSI
GHUGHLI
GOLA GOKARAN NATH
GONDA
GORAKHPUR
GOSHAINGANJ
GURSHAHAIGANJ
HAIDARGARH
HALDWANI
HANDIA KHAS
HARDOI
HARSAON
HASNAMGANJ
HARPALGANJ
HARPALPUR

Cawnpore Area (Contd.)

JAHANAGANJ ROAD
JAIS
JAKHANIA
JALALPUR
JANGHAI JN.
JANGIGANJ
JARWAL ROAD
JASWANTNAGAR
JAUNPUR
JEONATHPUR
JHANSI
JHINJHAK
JHUSI
KABRAI
KACHHLA GHAT
KACHHWA ROAD
KADIPUR
KAIMGANJ
KALPI
KAMALGANJ
KAMALPUR
KANAUJ
KANCHAUSI
KARARI
KARENGI
KARWI
KASGANJ
KASHI
KATHGODAM
KATHKUIYAN
KATKA
KAWAPUR
KEORIPUR
KERAKAT
KHADA
KHAGA
KHALILABAD
KHATEMA
KHETA SARAI
KHORASAN ROAD
KICHHA
KIDIHDAPUR
KOHNDAUR
KONRH ROAD
KOPAGANJ
KUNCH
KUNRAGHAT
KUSMHI
LACHHMIPUR
LAKHIMPUR KHERI
LAKSHMIGANJ
LALGANJ
LALITPUR
LAR ROAD
LUCKNOW
MADHOGANJ
MADHOSINGH
MAHMUDABAD
MAHOBA
MAHOLI
MAHPUR
MAINPURI
MAKRANDPUR
MALIHABAD
MALIPUR
MALLAWAN
MANAURI
MANDAH ROAD
MANKAPUR
MANPUR NAGARIA
MARRERA
MARIAHU
MASKANWA
MAU AIMA
MAU NATH BHANJAN
MAURANIPUR
MEJA ROAD
MIRANPUR KATRA
MIRZAPUR
MISRIKH TIRATH
MOGHAL SARAI
MOTH
MUHAMMADABAD GOHNA
MUNDERWA
MUSAFIRKHANA
NAGARIA SADAT
NAIPALGUNJ ROAD
NANDGUNJ
NANPARA
NAUTANWA
NAWABGUNJ GONDA
NIBKAROR
NOWGARH
ORAI
ORCHHA
PACHPERWA
PADRAUNA
PARIAWAN
PARTABGARH
PATIALI-ON-GANGES
PAYAGPUR
PEPPEGANJ
PHAPHUND
PHARENDA
PHARIHA
PHEPHNA
PHULPUR
PIPRAICH
PITAMBERPUR
PILIBHIT
PUKHRAYAN
PUNCH
PURANPUR
RAGAUL
RAGHURAJ SINGH
RAI BARELI
RAJA-KA-RAMPUR
RAJA TALAB
RAJAWARI
RAMKOLA
RANI-KI-SARAI
RASAULI
RASRA
RASULABAD
REOTI
RICHHA ROAD
RISIA
RUDAIN
RUDAULI
RURA
SAADAT
SAFIPUR
SAHATWAR
SAHAWAR TOWN
SAHJANWA
SAIDPUR BHITR
SAIYEDRAJA
SAKALDIHA
SALIMPUR
SANDILA
SARAIMIR
SARDARNAGAR
SARJU
SHAHGANJ
SHAHGARH
SHAHJAHANPUR
SHAMSHABAD
SHRIKHUPUR
SHIVPUR
SHOHRATGARH
SHRI KRISHNA NAGAR
SIDHAULI
SIKANDARPUR
SIKANDRA RAO
SISOTARGHAT
SISWA BAZAR
SITAPUR
SORON
SULTANPUR
SURAIMANPUR
SURIAWAN
TAHSIL DEORIA
TAHSIL FATEHPUR
TAKIA
TAMKOHI ROAD
TANAKPUR
TANDA
TARAON
TARIGHAT
TILHAR
TIKRI
TULSIPUR
UJHANI
UNAO
UNCHAULIA
USKA BAZAR
WALTERGANJ
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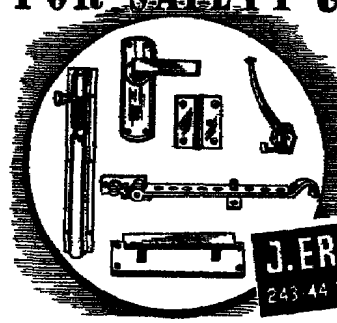
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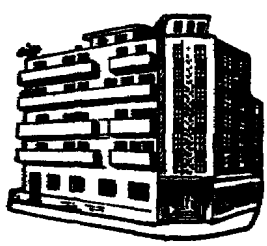
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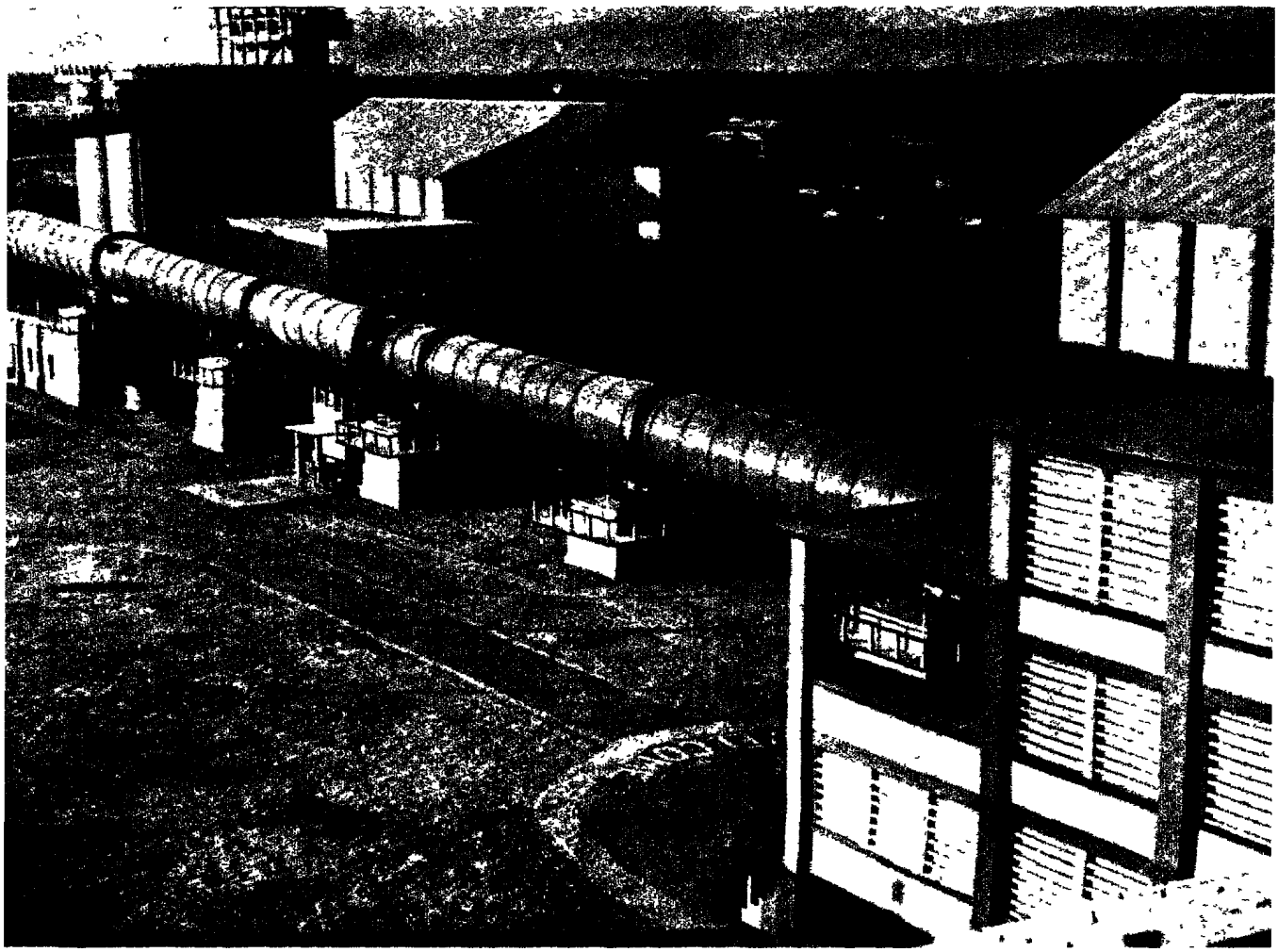
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1E CHAIBASA CEMENT WORKS AT JHINKPANI, BIHAR.

A general view of the Kiln and Slurry Silos.

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